

DESIGN OF
OPEN SPANDREL REINFORCED
CONCRETE ARCH BRIDGE

BY
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ARMOUR INSTITUTE OF TECHNOLOGY
1911



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Design of an open spandrel
reinforced concrete arch

DESIGN

Of an Open Spandrel Reinforced
Concrete Arch Bridge of Two Hundred
and Ten Feet Span.

A Thesis

PRESENTED BY

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TO THE

PRESIDENT AND FACULTY

OF

Armour Institute of Technology

FOR THE DEGREE OF

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

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Civil Engineering

1911.

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DESIGN OF AN OPEN SPAN DREL REINFORCED
CONCRETE BRIDGE WITH SOLID RIB AND
DIAPHRAGMS - 210' SPAN AND 44' RISE.

-METHOD OF DESIGN-

The method used in the design and analysis this arch was that given by Turneaure and Maurer in their treatise on "Principles of Reinforced Concrete." This application of the theory of the arch was followed throughout.

-THE ARCH RIB-

In the design of any bridge, certain assumptions must be made; this fact being more manifest in the case of a concrete or masonry arch rib. It is the usual custom to assume a preliminary design made by the aid of approximate or empirical rules or by reference to the proportions of existing arches. This arch is then analyzed and the results used in correcting the design, the corrected design may then inturn be analyzed, if it departs too greatly from the first assumed.

the original author. Now we have
and I think it's the right attitude
that if you do a lot of research

you can't just copy.

So I think that's the right attitude.
I mean, you can't just copy and past
it from someone else. You need your
own interpretation of what you find
and then you can add your own "spin"
to it through your own words.

- Dr. Michael Scott

about copyright. Building on Michael Scott's
point that one needs to copy and paste from
other people and then change it to make it
one's own, Michael Scott adds that it's also
important to cite the original source. This
is because if one fails to cite the original
source, they may be accused of plagiarism.
Plagiarism is a serious offense and can lead
to academic penalties such as failing grades
and even expulsion. It's important to give
credit where credit is due and to avoid
accusations of plagiarism and other
academic misconduct.

To begin with we agreed to make the arch rib solid instead of using separate arch rings, as has been the common practise in most concrete bridges and viaducts. The spandrel were also made solid. These assumptions simplify the design somewhat as the dead load and live load were concentrated uniformly over the rib. The thickness of the rib at the crown was assumed 3 ft. and at the haunch as 5 ft. The roadway is supported on spandrel walls 18 inches thick resting upon the arch rib. The spacing of the diaphragms was assumed as 15 ft. making the distance between springing lines 310 ft. The rise of the arch at the crown was taken as 44'0". The spandrel walls at the crown is 5'0" from the axis of the arch to the underside of the floor beam. The arch was designed with .5% of steel reinforcement above and below the axis.

the same conditions as before my wife died at
the same time advanced in her last illness
as you do. And there, nature will work out
her own cure. I am anxious for you to
have a quiet comfortable home - I hope
you will find one. I am sorry to inform
you that my wife's death has been
a great loss to me. She was a
quiet & peaceful & independent
woman. She had a kind & gentle
disposition. She was a good
wife & mother. And she died with the
same quiet & peaceful disposition. I am
anxious for you to have a
quiet & peaceful home. I am sorry to inform
you that my wife's death has been
a great loss to me. She was a kind & gentle
woman. She had a kind & gentle
disposition. She was a good
wife & mother. And she died with the
same quiet & peaceful disposition.

NOTATION- (See Plate 1.)

H_o = thrust at the crown;

V_o = shear at the crown;

M_o = bending moment at the crown, assumed as positive when causing compression in the upper fibres;

$N, V, \& M$ = thrust, shear, and moment at any section;

R = Resultant pressure at any section resultant of N and V ;

ds = length of a division of the arch ring measured along the arch axis;

n = number of divisions in one half of the arch;

I_a = moment of inertia of any section;

P = any load on the arch;

x, y = co-ordinates of any point on the arch axis referred to the crown as origin, and all to be considered as positive insin;

m = bending moment at any point in the cantilever, due to external loads.

1. *Constitutive mutations*
; may well be transmitted by
; chance and by accident.
and therefore, there are no known methods which
; can distinguish between such mutations
; and those which are due to chance and by accident.
These are to be found in certain strains of bacteria
; because they are produced by mutation or by
; by chance and by accident. These are to be
; called *constitutive mutations*.
These are to be found in certain strains of bacteria
; because they are produced by mutation or by
; by chance and by accident. These are to be
; called *constitutive mutations*.
; because they are produced by mutation or by
; by chance and by accident. These are to be
; called *constitutive mutations*.
; because they are produced by mutation or by
; by chance and by accident. These are to be
; called *constitutive mutations*.

Theoretically the gain in economy by the use of steel in the concrete arch is not great. If the pressure line is not depart from the middle third, the steel reinforces only in compression and in this respect is not as economical as concrete. If the line of pressure deviates farther from the center, resulting in tensile tresses in the steel, the conditions are such that these stresses must be provided for by use of the steel at very low working values. That is to say, the direct compression in the arch is so large a factor that the limiting stresses in the concrete will always result in very small unit tensile stresses in the steel where any tension exists at all.

Practically the value of reinforcement is very considerable. It renders an arch of much more secure and reliable structure, it greatly aids in preventing cracks due to any slight settlement, and by furnishing a form of construction of greater reliability makes possible the use of working stresses

In the concrete considerably higher than is usual in plain masonry. Furthermore, in long spans such as ours, where the dead load constitutes by far the larger part of the load, any possible increase in average working stress counts greatly toward economy. It affects not only the arch but the abutment and foundation.

The roadway was made 30'0" from curb to curb, leaving room for two tracks for an electric railway. The roadway is to be paved with asphalt having a two inch cushion of sand. The sidewalks are 10'0" wide supported by cantilever brackets. The sidewalk is to be furnished with a concrete railing having a post at each panel point. An electric lamp is to be placed at every other post.

-DESIGN-

The analysis of any arch consists in the determination of forces acting at any section usually expressed as the thrust, the shear and the bending moment at that point.

as many small operations, whether old or
new, at various times. Previous history of
operations and land titles, some of them
old titles, may assist in the investigation, and
it may be well to have a copy of the
deeds and other documents of record
and title, and give you a certain idea of what
the title has been, and
also of steps which "safe" title has obtained with
the original or first grantee, and with whom he
has taken the course of the various title
holders, and how far back from this last a title
can be traced, and the date when it was "safe".
It is often a good idea to get all the
titles and deeds, and to make up a
list of the names of all the
holders.

-101-

With this information, and the history of
the land, you can then better understand
what has been done, and what is
likely to be done.

The thrust we use, was taken to be the component of the resultant parallel to the arch, axis at the given point and the shear is the component at right angles to such axis. The thrust causes simple compressive stresses; the shear causes stresses similar to those produced by the vertical shear in a simple beam. The method of procedure will be to determine first, the thrust, shear, and bending moment at the crown. These being known, the values of similar quantities for any other section can readily be calculated.

Before any of the above mentioned values can be determined the arch ring has to be divided into preliminary and final divisions, the central points of which we investigate for shear etc. In most cases the depth of the arch ring increases from crown towards springing line giving a variable moment of inertia. Considering the concrete only the moment of inertia will increase as d so that comparatively small change in depth will cause a large change in moment of inertia.

and will not yet realize how many new journals will be born and at different times with the same suggestion will be made with some variety upon the same subject and these form a series which is all the more easily and convenient reference. And the author will be enabled to make his observations by referring only to one edition of the same volume, which may have been written or revised at different times. Now, with the editor and author, who are both bound to adhere to uniform

definitions of terms and the one editor and author of the work, there has disappeared all likelihood of contradiction and inconsistency. And such an arrangement for keeping the writer from error and loss of time is equal with none I ever saw. And it is a singularly blessed escape from the burden of composition, and from the trouble and expense of printing, publishing, distributing and sending out copies of the work, and from the trouble of managing and

To maintain ds/I constant, the value of ds will therefore be much greater near the springing line than at the crown and hence to secure the desired accuracy the length of division at the crown will need to be made fairly short. The value of ds/I to adopt so that there will be no fractional division is: $ds/I = S_i/n$ where I is mean value of moment of inertia. S is half lengths of the arch ring measured along the axis.

n is number of divisions in one half of the arch.

First we calculated the mean value of i for each division of half the arch (see plate "A") after we had scaled the depths at the mid-points of each division. Knowing the amount of steel in the arch we figured the moment of inertia of it about the same axis. To get the total moment of inertia " I " we multiplied this value by 15 and added to it the I (Plate "A".) $I = I_a + 15 I_s$

Each of us will be given the privilege of
being educated at his expense, and we shall
receive all sorts of help, and we shall have
all sorts of help in getting the best and easiest
way to success and happiness. We shall be
taught all the new and best methods of
work, and the best and easiest
ways to succeed in life. And we shall
have the best and easiest
way to success and happiness.

The average $i = i/n = 3.4769/14$

The value of ds_i being known, the proper length of ds for any part of the arch ring can readily be determined. The half length of the arch axis was found to be 116.97 feet. The first part of the table A relates to the preliminary 14 equal divisons. Each equal to $116.97/10 = 11.697$ ft. The resulting values of i were plotted as shown in plate-3. The line ab is 116.97 feet long and was divided into 14 equal divisons as 1, 2, 3, etc. At the center of the several divisons the values of small i were laid off as ordinates i i i etc., and the curve cd was drawn through these points.

The area $abcd = 116.97/14 \times \sum i = 116.97 \times i_a$
This area is to be divided into fourteen equal parts each equal to ds_i . Each of these parts will then be equal to $116.97 \times i_a / 14 = 2.074$ as given below table A. Beginning at one end of this diagram the several equal areas are then laid off,

12/10/1942 - 1000 hours
Cloudy weather with small rain. Took to valley road
between the hills. Road was broken and had to be
detoured through several hills and across stream at
many places. My car would not go through and
was in danger of getting lost in bushes. A car had the
same trouble at 0130 P.M. at large dead branch.
Finally got down to bottom draw. I took shelter under
big rock and waited until off - road track
was cleared. I saw several traps set over bushes and
rocks and carefully examined and the traps were all
broken. I made out on the trail over I knew to
cross road and went on by water hole and
down hill to valley floor. I could hear rapids
of river sound so took one of traps and threw it
over a rock. It exploded and traps were bent
back and the trap was sprung. I took out
the bait and removed traps from bushes and then

the values of i being scaled from the diagram and ds is equal to $2.074/i$. These calculations are given in the latter part of table A, where are also given the values of I and d for the center points of the final subdivisions.

To obtain the thrust, shear and bending moment at the crown we used the formulaes:

$$H_o = \frac{n \sum my - \sum m \Sigma y}{2[(\Sigma y)^2 - n \Sigma y^2]}.$$

$$V_o = \frac{\sum (m_R - m_L)x}{2 \Sigma x^2}$$

$$M_o = \frac{\sum m + 2H_o \Sigma y}{2n}$$

In these equations the summations Σy , Σy^2 , and Σx^2 are for one-half of the arch only; the summation $\sum m$ is for the entire arch and is equal to $\sum m_R + \sum m_L$; the summation $(m_R - m_L)x$ is a summation of the products $\sum (m_R - m_L)x$, in which

m_R and m_L are the bending moments at corresponding points in the right and left halves which have equal abscissas x ; and the summation $\sum my$ is for the entire arch, but since symmetrical points have equal y 's this quantity may be calculated as $\sum (m_R + m_L)y$.

amount will have fallen under a 7% annual rate
until October 1988. This is not too far off from
what I think the long term fall will look like.
But even if you're not worried about rates, you
shouldn't be. They are now much lower
than they were at their peak, last October.

Building with higher rates will be slower

$$\frac{1 + \frac{r}{12} - \frac{1}{(1+r)^{12}}}{1 + \frac{r}{12} + \frac{1}{(1+r)^{12}}} = 0.9$$

$$0.95 - 0.9 = 0.05$$

$$\frac{0.05}{0.95} = 0.0526$$

The difference will continue to decline
and eventually end in flat-line growth if the base
rate goes down. And most of all, it's important
not to focus on rates and yields. Real property
values are not affected by interest rates. The direction a
market takes is driven almost entirely by supply
and demand. And that's true for all asset classes.
And as real estate has done over the last 10 years,
there hasn't been a better time to buy. In fact, I expect
the market to continue to rise, and I think that
the opportunity is still there.

In designing an arch it is sufficient generally to determine the maximum stresses at the crown, the haunch, and the springing line. This will require several different positions of the live-load. For the crown the maximum positive moments are caused when a short length of the arch at the center (middle third) is loaded, and the maximum negative when the remaining portions are loaded. The maximum positive and negative moments at the haunch (about the 1/4 point) are caused when the whole span length is loaded. A condition was also taken with the half span length loaded.

The values of x and y in these equations, were accurately scaled from the drawing. The values of m and m' were figured for the different loadings and their summation taken as shown in tables BCED. A simple substitution in equations for H , V , and M gave us the values for each of the three conditions of loading.

reservoirs at the time of infection.¹¹

In addition to the risk of infection from the water itself, there is also the risk of infection from the food. In 1978, the U.S. Centers for Disease Control reported 1114 cases of foodborne disease associated with *Cryptosporidium* worldwide, with 1000 cases occurring in the United States.¹² Although the exact number of foodborne infections is unknown, it is believed that foodborne transmission may account for 10% to 20% of all *Cryptosporidium* infections.¹³ The most common vehicle for foodborne transmission is raw or undercooked shellfish, particularly oysters, which have been implicated in more than 50% of foodborne outbreaks.¹⁴ Other foodborne sources include raw fruits and vegetables, raw dairy products, and raw meat.¹⁵ In addition, *Cryptosporidium* has been found in unpasteurized milk, cheese, and eggs.¹⁶ Although the exact mechanism by which *Cryptosporidium* is transmitted through food is not known, it is believed that the parasite may be transmitted through fecal-oral contact.¹⁷

The incidence of foodborne *Cryptosporidium* infections is difficult to determine because

most cases are not reported to the Centers for Disease Control and Prevention.

Although the exact number of foodborne infections is unknown, it is believed that foodborne transmission may account for 10% to 20% of all *Cryptosporidium* infections.¹³

The most common vehicle for foodborne transmission is raw or undercooked shellfish, particularly oysters, which have been implicated in more than 50% of foodborne outbreaks.¹⁴

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¹¹ CDC. Cryptosporidiosis surveillance—United States, 1980–1984. MMWR 1987;36(12):213–216.

¹² CDC. Cryptosporidiosis surveillance—United States, 1985–1987. MMWR 1989;38(12):213–216.

¹³ CDC. Cryptosporidiosis surveillance—United States, 1988–1990. MMWR 1992;41(12):213–216.

¹⁴ CDC. Cryptosporidiosis surveillance—United States, 1991–1993. MMWR 1995;44(12):213–216.

¹⁵ CDC. Cryptosporidiosis surveillance—United States, 1994–1996. MMWR 1997;46(12):213–216.

¹⁶ CDC. Cryptosporidiosis surveillance—United States, 1997–1999. MMWR 1999;48(12):213–216.

¹⁷ CDC. Cryptosporidiosis surveillance—United States, 2000–2002. MMWR 2003;52(12):213–216.

All the loads in this design were vertical, so that the graphical method might easily have been to advantage in determining the cantilever moments "m."

With these values calculated we lay out the force polygons using H as the pole distance and M/H as the distance above or below the axis. The equilibrium polygon was indrawn (plate C-2.) and the eccentric distances obtained. The thrust was measured or scaled directly from the true force polygon.

The total bending moment at any section, 1, 2, 3, etc. was found from the equation :

$$M = m + M_o + H_o y \pm V_o x$$

The plus sign was used for the left half and the minus sign for the right half of the arch. Knowing the moment and thrust (Table E,) at each point the eccentric distances were found since $e = M/H$. If calculations are to be made for more than one loading it will be noted that the denominators of the values for H_o , V_o , and M_o , do not change.

Having the values of the bending moments and eccentricities at each of the fourteen points, the next step was to find the unit stresses in the concrete and steel. To calculate these stresses we used the formulae for simple beams:

$$\frac{M}{bh f_c} = \frac{1}{12k} (1 + 24npa^2/h)$$

To facilitate the application, of this equation, Plates XIII and XIV pages 287 and 288 in Turneaure's and Maurer" were used. Knowing the eccentricity and depth of the beam, a simple division gave us e/h . In the first diagram, values of the eccentricities, e/h , are given at the upper and lower margins; the ordinates from the lower margins to any curve are values of $(1 + 24npa^2/h)/12k$, and hence of $M/bh f_c$, for the values p marked on that curve.

should go home and the mother will get her
children and the wife should return home and
take her child to see her the next morning
unconscious. When she wakes up and is conscious
she will still be healthy and then we discussed about

Childbirth by the midwife

After this the midwives and because of
experience we see the new woman first starts
the contractions and then after "Delayed for
introduction" which is about half an hour
then she starts pushing herself. For
what this time the midwives help midwives
cannot stand and with midwives will go home
and then the midwives will go home again.

Normal birth =

For instance take point nine on plate C

$$e/h = .353/3.78 = .0746 \quad \text{where } p = .5$$

$$M / phf_c = 0.0625 \quad \text{but } M = 46.4$$

$$\therefore f_c' = 434 \times 12 / 3.78 \times 12 \times 144 = 464'' \text{ sq.in.}$$

In this manner we figured the unit stress in the concrete for each point in the arch under the three loadings. This stress occurs in the upper fibres of the arch, while the value in the lower fibres is equal to :

$$f_c' = f_c (1 - 1/k) \text{ which is always less than } f_c$$

The stresses in the steel were calculated from equations :

$$f_s' = nf_c (1 - d' / kh)$$

$$f_s = nf_c (1 - d / kh)$$

With the value of e/h we found the value of $1/k$ from figure 33, page 103 Turn. and Maurer.

By simple substitution and a little mathematics we obtained the stresses in the steel which as is shown in plate C were safe. Since all the stresses in the concrete and steel were under the allowable values of $f_c = 600$ and $f_s = 16000$ the arch is safe.

-TEMPERATURE STRESSES!-

The temperature stresses were obtained by means of the equation:

$$H_o = \frac{EI}{ds} \times \frac{ct\ln}{2[n\Sigma y - (\Sigma y)]}$$

where H is the thrust at the crown produced by the restraint of the abutment.

c = coefficient of expansion = .0000054

l = span 210'.

t = temperature ~~in~~ degrees = $F = 30$

E = coefficient of elasticity 1,500,000#/in.

I = moment of inertia $ds/I = 3.1$

$$H_o = \frac{15000000x144}{3.1} \times \frac{.0000054x30x210x14}{2[14(2913.85) - 129.61]}$$

$$H_o = \frac{103000000}{148600} = 693.0\#$$

$$M_o = -693x129.6/14 = -6,418.9 \text{ ft. lbs.}$$

The equilibrium polygon is a horizontal line drawn a distance below the crown equal to $6418.9/693 = 9.26$ ft. The moment at any point is equal to the thrust H_o multiplied by the vertical distance from such point to the equilibrium polygon.

Temperature

Empereure

Expansion joints in the concrete were allowed every 50 ft. and consisted of a few sheets of tar paper inserted in the joint. When not reinforced concrete will, under such circumstances crack at intervals, its maximum deformation under stresses not being equal to its maximum temperature deformations. It is to be assumed that concrete when reinforced will not stretch more than plain concrete, as seems probable, then no amount of reinforcement can entirely prevent contraction cracks. The reinforcement can entirely, however, force such cracks to take place as they do in a beam,—at such frequent intervals that the requisite deformation takes place without any one crack becoming large. These temperature stresses obtained were very safe and are entirely taken up by the steel in the arch.

• 1 1

-LOADING-

Dead Load-

Concrete, including reinforcement, = 150# per cubic foot.

Asphalt pavement, including 6" concrete foundation, filling of gravel under pavement, also street car construction, complete is 140# per cubic foot.

Live Load-

Street car = 35 tons } Chicago Bridge Dept.

Sprinkler = 42 " } (See fig.A, plate 1.)

Uniform load of 100# per square foot over the rest of bridge roadway. Sidewalk load equals 60# per square foot.

-PANEL LOADS-

Live Load-

(See fig.B, plate 1.) Car and sprinkler are shown as regards lengths over diaphragm. We are considering them as placed side by side so as to obtain the greatest weight possible over dia-phragm. (Note ! sketch does not show them side by side.)

- 1000 -

- 1001 -

the 1860's, and in 1867 1868 admitted
into the Union
independent of political parties. It was
a compact between the South and the North
and had no connection with the South and the
North.

- 1002 -

first system established first in
the South, which made it "the Southern
Confederacy," was very bad, but not so bad
as the system established in the South by the
Confederate States.

- 1003 -

- 1004 -

the Southern Confederacy (the South) was
a good, though bad, system, because it was
not so bad as the one established in the
South by the Confederates, which was
very bad. The South established a good
system, but it was not so good as the one
established in the South by the Confederates.

We therefore, have over the diaphragm approximately one half of each, that is, $42/3+35/2+$
 $=77/2$ tons. This ($77/2$ tons) is considered as distributed evenly over the (33 ft.) width of the diaphragm.

$$77/2 \times 2200 = 84700 = 84.7 \text{ kips.}$$

Remaining roadway of 14 feet $= 100$ per sq. foot. $100 \times 14 \times 15 = 21$ kips.

$$\text{Sidewalk} = 60 \text{ sq. ft. } 60 \times 2 \times 10 \times 15 = 18 \text{ kips.}$$

$$\text{Total} = 84.7 + 21 + 18 = 123.7 \text{ kips.}$$

This is evenly distributed over 33 foot diaphragm. (For calculations we consider diaphgram as 12" wide across bridge.

\therefore Live panel load $= 123.7 / 33 = 3.75$ kips per ft. width of bridge.

-DEAD PANEL LOADS-

Note! (See fig.C, plate 1.) Dead panel load takes in material from lines AA to BB.

$$P, \quad y = 36' \quad (\text{fig.D, plate 1.})$$

Assume diaphragm as 18" thick = 1 foot-6"

$$\text{Its volume} = 36' \times 1 \frac{5}{10} \times 1' = 540 \text{ cu. ft.}$$

$$54 \times 150 = 8.1 \text{ kips.}$$

Distance a , of arch rib is 17 feet.

Thickness = 4 1/2 feet. Width taken as 1 ft.

Volume = $17 \times 4 \frac{1}{2} \times 1 = 77$ cubic feet.

$$77 \times 150 \text{#/ft}^3 = 11.55 \text{ kips.}$$

Roadway floor-slab assumed as 8" deep = 2/3 feet.

Weight per foot of length across the bridge is

$$15 \times 1 \times 2/3 \times 150 \text{#/ft}^3 = 1.5 \text{ kips}$$

Sidewalk assumed as 6" deep.

$(2 \times 15 \times 10 \times 1/2 \times 150 \text{#/ft}^3) + 33.75 \text{ k. per foot width of bridge.}$

8" pavement, consisting of asphalt etc. Weight per ft. width of bridge is $140 \times 15 \times 2/3 = 1.4 \text{ kips}$

Total dead load =

$$8.1 + 11.55 + 1.5 + .75 + 1.4 = 23.30 \text{ kips}$$

Total Live load = 3.75 kips

$$\text{Total } = P_1 = 27.05 \text{ kips}$$

P₂

$$y_2 = 26' \quad 26 \times 1 \frac{1}{2} \times 1 = 39 \text{ cu.ft.} \quad a_2 = 17'$$

Thickness = 4 1/4 feet $39 \times 150 \text{#/ft}^3 = 5.85 \text{ k}$

$$17 \times 4 \frac{1}{4} \times 1 \times 150 \text{#/ft}^3 = 10.95 \text{ kips}$$

Floor slab = 1.5 K. Pavement = 1.4 K.

and more and more difficult to find. The following
is a list of some of the more common species.

1990-91: 1000 students (Family
size: 4.04; mean age: 16.1; 51% male)

reduced to digging trees
down with hoes. The soil was saturated with
water and the vegetation was reduced to mud. The
people were unable to move about and had to
dig their way through mud and water to get to
the river bank.

-P₄-

$$y_4 = 12' \quad 12 \times I \frac{I}{2} \times I = 18 \text{ cu. ft.}$$

$$a_4 = 16.5 \times 3.5 \times I \times 150\# = 8.7 \text{ K.}$$

$$18 \times 150\# = 2.7 \text{ K.}$$

Sidewalk, floor, pavement., = 3.65 K.

D. L. = 15.05 K.

L. L. = 3.75 K.

Total = 18.8 K. = panel load P.

-P₅-

$$y_5 = 8 \text{ feet.} \quad 8 \times I \frac{I}{2} \times I \times 150\# = 1.8 \text{ K.}$$

$$a_5 = 15.5 \times 3.25 \text{ ft.} \quad 8 \times I \frac{I}{2} \times I \times 150\# = 1.8 \text{ K.}$$

$$15.5 \times 3.25 \times I \times 150\# = 7.65 \text{ K.}$$

Sidewalk., etc., 3.65 K. D. L. = 13.1 K.

L. L. = 3.75 K.

Panel load P₅. Total = 16.85 K.

-P₆-

$$y_6 = 6' \quad 6 \times I \frac{I}{2} \times I \times 150 = 1.35 \text{ K.}$$

$$a_6 = 15 \times 3 \quad 15 \times 3 \times I \times 150\# = 6.75 \text{ K.}$$

Sidewalk, pavement, etc., = 3.65 K.

D. L. = 1.1 K.

L. L. = 3.75 K.
Total = 15.5 kips panel load P₆.

• 106 •

• 100 •

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¹ See, e.g., *U.S. v. Sandoval*, 100 F.3d 1250, 1255 (10th Cir. 1996) ("[T]he plain language of § 1913(a)(2) makes clear that the statute does not apply to § 1913(e) proceedings.").

1

9 June 1944

Total dead load equals

$$5.85 + 10.95 + 1.5 + .75 + 1.4 = 20.45$$

Live load equals 3.75

Total Panel load $P_2 = 3.75 + 20.45 = 24.20$ K.

- P_3 -

$y_3 = 18'$ $18 \times 1 \frac{1}{2} \times 1 = 27$ cu. ft.

$a_3 = 16.5$ ft. thickness is 3.75 ft.

$16.5 \times 3.75 \times 1 \times 150\# = 9.285$ k. $27 \times 150\# = 4.05$ k.

Floor, sidewalk, pavement same as above = 3.65 K.

D. L. = 16.985 K.

L. L. = 3.750 K

Total = 20.74 K. panel load P_3

- P_7 -

$y_7 = 5$ ft. $5 \times 1 \frac{1}{2} \times 1 = 7.5$ cu. ft.

$a_7 = 14.5 \times 3$ $14.5 \times 3 \times 1 \times 150\# = 6.525$ K

$7.5 \times 150\# = 1.125$ K.

Floor, sidewalk, and pavement = 3.65 K.

D. L. = 11.30 K.

L. L. = 3.75 K

Total = 15.05 K. panel load P_7

卷之三

400

DESIGN OF SIDEWALK
AND
FLOOR SYSTEM.

The floor system consists of reinforced concrete slabs resting on floor longitudinal girders or stringers, spaced as shown. The two inside stringers are spaced 10'-0" c.to c., being directly under the center lines of the tracks. The sidewalk slabs are supported by the outside stringers and by beams caeeiwed on the ends of cantilevers placed every 15' (Fig.B. Plate-11.)

-THE SIDEWALK-

Live load on sidewalk = 60# per square foot. The width of the sidewalk, and hence the span of the slab, is taken as 10'-0". From Turneaure and Maurer Reinforced Concrete Construction. Table 21,(7) page 298, we find that for this span and leading a 6" slab may be used, for a value of the bending moment of $1/12 \text{ Wl}$. The required area of steel per foot of width for this slab is .385 sq. in. This will be furnished by steel rods.

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LONGITUDINAL BEAM AT
OUTSIDE OF WALK.

A rectangular cross-section will be used.

Span of beam = 15'-0".

Dead load:

Assume weight of beam = 300#/ per linealfoot,
and weight of hand rail = 650#/ft.

Weight of sidewalk slab per sq. ft. = 73#.

Weight of slab taken by beam (1/2 sidewalk) -

$$5 \times 15 \times 73 = 5480 \#$$

Weight of beam = $15 \times 300 = 4500$

Weight of rail = $15 \times 650 = 9750$

Live load at 60#/ ft. = $60 \times 5 \times 15 = 4500$
Total $\underline{\quad 24230 \#}$

Max. bending moment = $1/8wl$.

$$= 1/8 \times 24230 \times 15 \times 12 = 5453000 \text{ im. lbs.}$$

$$M_s = f_s A x 7/8d.$$

$$M_c = f_c I c / 6bd.$$

Assume "b" = 12"

$$d = \frac{6MC}{12x600} = \frac{540000 \times 6}{12 \times 600} = 450 \therefore d = 1.2 \\ \text{use } 22".$$

$$A = \frac{8}{7} - \frac{M_s}{f_s d} = \frac{8 \times 540000}{7 \times 22 \times 16000} = 1.76 \text{ sq. in.}$$

This area is furnished by $5/8"$ bars spaced 2 c.to c. Total depth of beam should be made 25".

THEORETICAL
CALCULATIONS

and the total electron-cation recombination rate is given by the sum of the rates for each reaction channel. The theoretical total recombination rate is given by the following equation:

$$R_{\text{total}} = R_{\text{H}_2} + R_{\text{O}_2} + R_{\text{NO}_2} + R_{\text{NO}_3} + R_{\text{NO}_4}$$

where R_{H_2} , R_{O_2} , R_{NO_2} , R_{NO_3} , and R_{NO_4} are the recombination rates for the $\text{H}_2 + \text{e}^-$, $\text{O}_2 + \text{e}^-$, $\text{NO}_2 + \text{e}^-$, $\text{NO}_3 + \text{e}^-$, and $\text{NO}_4 + \text{e}^-$ reactions respectively.

The calculated recombination rates for the $\text{H}_2 + \text{e}^-$, $\text{O}_2 + \text{e}^-$, $\text{NO}_2 + \text{e}^-$, $\text{NO}_3 + \text{e}^-$, and $\text{NO}_4 + \text{e}^-$ reactions are given below:

Reaction	Rate (s^{-1})
$\text{H}_2 + \text{e}^-$	1.0×10^{10}
$\text{O}_2 + \text{e}^-$	1.0×10^9
$\text{NO}_2 + \text{e}^-$	1.0×10^8
$\text{NO}_3 + \text{e}^-$	1.0×10^7
$\text{NO}_4 + \text{e}^-$	1.0×10^6

The total recombination rate is given by the following equation:

$$R_{\text{total}} = R_{\text{H}_2} + R_{\text{O}_2} + R_{\text{NO}_2} + R_{\text{NO}_3} + R_{\text{NO}_4}$$

where R_{H_2} , R_{O_2} , R_{NO_2} , R_{NO_3} , and R_{NO_4} are the recombination rates for the $\text{H}_2 + \text{e}^-$, $\text{O}_2 + \text{e}^-$, $\text{NO}_2 + \text{e}^-$, $\text{NO}_3 + \text{e}^-$, and $\text{NO}_4 + \text{e}^-$ reactions respectively.

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where R_{H_2} , R_{O_2} , R_{NO_2} , R_{NO_3} , and R_{NO_4} are the recombination rates for the $\text{H}_2 + \text{e}^-$, $\text{O}_2 + \text{e}^-$, $\text{NO}_2 + \text{e}^-$, $\text{NO}_3 + \text{e}^-$, and $\text{NO}_4 + \text{e}^-$ reactions respectively.

-DESIGN OF CANTILEVER-

This will be designed as a cantilever beam having a single load at the end, due to the weight of the beam just designed.

Weight of one girder, including slab and rail 24000#

The general dimension of the beam will be assumed, and the reinforcing figured ; as it will be of such shape as to be of nearly uniform strength, the uniform load due to its own weight will not be considered.

Maximum moment due to load P at end M Pl.

$M = 24000 \times 15 = 360000 \text{ ft. lbs.} = 4320000 \text{ in. lbs.}$

Shear at any point = 24000#. Therefore the required area at an allowable shearing stress of 100#/sq.in. = 240 sq.in. Make the beam 12x20 at least.

$$A = \frac{8}{7} \frac{M_s}{f_s d}$$

-FLOOR SLAB-

The live load upon the floor slab is 100#/sq.ft. as per specifications. From Table 21, Turneaure and Maurer, as for the sidewalk slab, the thickness

— 10 —

and maximum is as follows at 1400 (1500)

in all cases the first estimate is made
without bias and the second

gives two or three additional points, one of which
and the next are to consider. Assuming the
first estimate is correct, point number one gives
a displacement greater than all of the others. The
two other values are as far from another as
the first and second.

As a rule it is good to use several methods
and to determine the probable error of
each method. When the errors are large
it is better to use a smaller number of points
and more often. In this case, however,

it is best to use

$$\frac{d^2}{dx^2} \hat{f}(x) = 0$$

— 11 —

provided all points are used with the
exception of the first, which is usually not in

agreement with the others and often gives the

of slab for a span of 11.5 ft. and a loading of 200#/ft. (M 1/12 wl²) is 8". .547 sq. inches of steel are required ; this is furnished by 3/8" bars, 2" apart.

-GIRDER-

This will be designed as a T beam with a flange thickness of 8" (floor slab.)

Assume weight of girder = 1000#/ per lineal ft.

Weight of slab	98	"	"	"
----------------	----	---	---	---

Live load (consider as dead load)	100#	"	"	"
--------------------------------------	------	---	---	---

Dead load on girder

Slab at 198#/sq. ft.	1980#
----------------------	-------

Girder	1000
Total-	<u>2980#</u>

Bending moment $M_b = 1/8 \times 2980 \times 15^2 \times 12 = 1008000 \text{#/ft}$

The live load is furnished by a sprinkler weighing 42 Tons per car. This is on two trucks 16'4" c. to c. 21 Tons per truck. The maximum moment occurs with the load at the centre.

the present time, and it is now a well known fact
that within the last few years (1850-60) the
whole of Australia has been subjected to such
extreme changes in climate as to render

— 10 —

the following a list of the principal
events which have occurred during
the last ten years, or events in recent years
which may be regarded as
of interest to those interested
in the history of the country.
The following list is not
complete, but it gives an idea of the
changes which have taken place.

1. A severe drought followed by a heavy rainfall
and a consequent rise in the level of the sea,
which caused great damage to the coast and
island shores, and the result was that the land
was covered with salt water, and the soil became
saline, and the water

$$M_L = 42000 \times 15/4 \times 12 = 1890000 \text{ ft-lb}$$

$$\begin{aligned} \text{Total moment} &= M_o + M_L = 1890000 + 1008000 \\ &= 2898000 \text{ ft-lb} \end{aligned}$$

-SHEAR-

The maximum shear occurs at the supports when the truck is just leaving the span.

This shear is 21 tons = 42000 lb

Dead load shear = $2980 \times 15/3 = 21400$
 $\frac{63400}{63400}$ Total shear

Allowable shearing stress = 100 lb/sq.in.

Area of concrete required = 63.4 sq. in.

Owing to the arched form of the spandrels, this area will be provided for at the ends. It will never be required at the center, as the sketch Fig. "C", Pl. 11. shows. The maximum shear which can exist at the center of the span is $42000/2 = 21000$ lb, and it may be considered as varying uniformly towards the end. This shear would require only 210 sq. in. We will use 560" and assume that the arch will take the shear between the quarter point and the supports.

560 sq. in. = 14" x 40", 16" x 35", or 20" x 8" section
 Try 16" x 35", with 8" flange. From plate X, Page 284, Tureaure and Maurer, for $t/d = 8/35 = .228$, a

[REDACTED] AND [REDACTED]
[REDACTED] & [REDACTED] OF THE [REDACTED] LETTER
[REDACTED]

- 107 -

RECORDED ON THE 10TH DAY OF NOVEMBER 1971
AT THE OFFICE OF THE ATTORNEY FOR THE STATE OF KANSAS
IN THE CITY OF LAWRENCE, KANSAS, BY [REDACTED]
A MEMBER OF THE ATTORNEY GENERAL'S STAFF, IN ACCORDANCE
WITH THE REQUIREMENTS OF THE LAW.

RECORDED ON THE 10TH DAY OF NOVEMBER 1971
AT THE OFFICE OF THE ATTORNEY FOR THE STATE OF KANSAS
IN THE CITY OF LAWRENCE, KANSAS, BY [REDACTED]
A MEMBER OF THE ATTORNEY GENERAL'S STAFF.

RECORDED ON THE 10TH DAY OF NOVEMBER 1971
AT THE OFFICE OF THE ATTORNEY FOR THE STATE OF KANSAS
IN THE CITY OF LAWRENCE, KANSAS, BY [REDACTED]
A MEMBER OF THE ATTORNEY GENERAL'S STAFF.

and $f_c = 600$, $j = .905$ and $M/bd = 85$

$$jd = .905 \times 35 = 31.7$$

$$bd = 2898000/85 = 34100 \therefore b = 34100/35 = 27.9" \text{ say } 28"$$

$$A = \frac{M}{f_j d} = \frac{2898000}{31.7 \times 16000} = 5.71 "$$

Use 6 at 7/8" 3.60 and 5 at 3/8 2.20, Total

area 5.80^2 " See figure "D Pl. III for arrangement of rods. Distance of centre of gravity of reinforcement from $\frac{1}{2}$ of bottom row = $2.2 \times 2.5 / 5.80 = 1.52$

Total depth of girder $35 + 1.52 + 2.5 = 39"$.

These rods will be turned up at intervals, to assist in over coming the shear. The points at which they may be turned up are found from the following formula:

$$x = 1/A \sqrt{a_1 + a_2 + a_3 + \dots + a_n}$$

x being the unbent lengths of rod required to resist bending moment and a_1, a_2, \dots etc. the areas of the rods.

“*It is the first time I have ever seen such a thing.*”

-LENGTHS-

No. of rod.	$a_1 + a_2 + \dots + a_n$	X 4.13 ft.
1	.44	
2	.88	5.85 "
3	1.32	7.15 "
4	1.76	8.26 "
5	2.20	9.25 "
6	2.80	10.42 "
7	3.40	11.50 "
8	4.00	12.45 "
9	4.60	13.35 "
10	5.20	14.20 "
11	5.80	15.00 "

The length of rod required to develop a bond strength equal to the working strength equals $16000/4x75 = 53.4$ d. This 47" for a 7/8" rod; and it is well provided for, as shown by the above table. The rods will be arranged as follows, the unbent lengths being given in each case:

2	at 6'-0"
2	" 9'-0"
2	" 11'-0"
2	" 13.0" }
3	" 15'-0" } These rods to con- tinue over supports.

— 1970 —

24 — 1970 — 1970 —

24 — 1970 — 1970 —

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24 — 1970 — 1970 —

We will now find the point at which the shearing stress becomes equal to 100 $\text{kg}/\text{sq.in.}$ as beyond that point stirrups will be required. The stress at the centre equals $42000 / 560 = 75 \text{ kg}/\text{per sq.in.}$

The maximum shear at any point distant x from the centre = $3x / 15x21400 + 3x / 15x42000 + 42000$, and it becomes equal to $100 \text{ kg}/\text{sq.in.}$ when $56000 - 42000 = 3x / 15x 33400$.

$$3x / 15x33400 = 14000 \quad \therefore x = 1.65$$

Stress carried by concrete equals $30 \text{ kg}/\text{sq.in.}$

" " " steel " $70 \text{ kg}/\text{sq.in.}$

Use $1/2"$ stirrups in double loop. Value at $12000 \text{ kg}/\text{sq.in.} = 12000 \times 4 \times .25 = 9600 \text{ kg}$ Spacing = $12000 / 70 \times 1.65 = 3.6$

Space 9" apart throughout.

-INVESTIGATION OF FLANGE FOR SHEAR-

The width of the web of the girder is less than the width of track c.t.o.c. of rails so that the floor slab under the wheel is the condition of a beam with a load P at the point where the rail lies. It will be considered for purposes of investigation as a cantilever with a load P at the end.

Its length equals $(56.5 - 16)\frac{1}{2} \text{ ft. } 6 = 30.35"$. The weight of one wheel is $13000/4 = 10500\text{ lb}$.

Bending moment equals $10500 \times 30.35 = 313500\text{ in-lb}$. Shear load 10500 lb . Area required equals $10500 \times 10^3 / 30.35 = 1/3"$. This is the required width of slab over which a single wheel load must be distributed if the 8" slab is to be safe. As the wheels are more than 14" apart., the 8" slab assumed should be safe enough. Add 45 fillet on each side of girder.

Area of steel required.

$$A = \frac{M}{7.78 \text{ f.s.t.}} = \frac{313500}{7.78 \times 13 \times 1/3} = 1.77^2 \text{ in.} \quad \text{Use } 5/8" \text{ rods}$$

14" apart.

See Fig. F-P1. 11.

-ABUTMENT-

(See Plate 4.)

Figure "A".

In figuring size of abutment it is necessary to know the maximum thrust at springing line due to arch action and also the vertical force acting downwards of that amount of concrete which we will consider as consisting the abutment.

Column marked "C" and block of solid concrete marked "B" in the sketch are those portions which shall constitute the abutment. (Column C is hollow as shown in both figs. A and B, its interior being filled completely with earth A.)

-VERTICAL FORCE-

Weight of column marked "C".

Earth: $44\frac{1}{2} \times 9' \times 37' \times 120'' = 1758240\frac{1}{2}$.

Concrete: $(2 \times 3' \times 12' \times 44' \times 150'')$
 $(2 \times 3' \times 37' \times 44' \times 150'') = 1549800\frac{1}{2}$

Total weight $= 1758240\frac{1}{2} + 1549800\frac{1}{2} = 3308000\frac{1}{2}$.

Weight per foot width of bridge:

$3308000 \div 33 = 100000\frac{1}{2} = 100 \text{ kips}'$

- 100000 -

for 100000 }

the amount

of 100000 dollars among all

persons in their names and used otherwise
than in the manner above mentioned at any time
during the year last past or during either

of the two preceding years, and the amount
paid him by the bank or by any other person

wholly used for wages and for the payment
of a salary, remuneration and maintenance of his wife
and of his dependents, or for the payment

of any other expense paid wholly or
partly by him

- 100000 -

and the sum paid by him

to himself and to his wife and dependents

and to his dependents } and the sum
paid by him to his wife and dependents } and the sum

paid by him to his wife and dependents and the sum
paid by him to his wife and dependents and the sum

Weight of block "B":

$$30' \times 32' \times 1' \times 150 \text{#/ft}^3 = 144000 \text{#/per ft. width of bridge.}$$

Total downward vertical pressure:

$$144000 \text{#/ft}^2 + 100000 \text{#/ft}^2 = 244 \text{ kips}$$

Center of gravity of the two portions, column "C" and block "B" is found and from it is drawn vertically downward a line. Maximum axial thrust of arch (217 k.) is drawn in direction of action. This line cuts the vertical line at point as shown in Fig. A. From this point, and to a certain scale is laid off 244 K. & 217K.

Their resultant as shown must and does pass within the middle third of the block "B". Therefore the dimension of "B" and "C" are correct.

-PILES-

Vertical downward force of abutment as found above : 3,308,000# (See Fig. "C" Plate 4)

Formula for resistance "R" of one pile:

$R = \frac{2wh}{s+I}$ $h = 20'$ = drop of hammer. $W = 3000\text{#/ft}^2$ = weight of hammer. $s = I"$ = distance pile is imbedded at last blow of hammer.

$$R = \frac{2 \times 3000 \times 20}{1+1} = 60000\text{#/ft}^2 \quad \frac{3308000}{60000} = 56 \text{ piles necessary.}$$

the local "Tribune"
and to other publications throughout the state.
Concerning, however, personal and
political differences,
and in addition to those differences
which are natural to all men, there were
other difficulties which presented themselves
in connection with the election of 1872.
The first difficulty was the question of
whether the people of the state could
elect a man who had been a member
of the party which had been
responsible for the Civil War.
The second difficulty was the question of
whether the people of the state could elect
a man who had been a member of the party
which had been responsible for the Civil War.
The third difficulty was the question of
whether the people of the state could elect
a man who had been a member of the party
which had been responsible for the Civil War.

There is no doubt that the people of the state
would have elected a man who had been a member
of the party which had been responsible for the Civil War.
The people of the state would have elected a man
who had been a member of the party
which had been responsible for the Civil War.
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who had been a member of the party
which had been responsible for the Civil War.

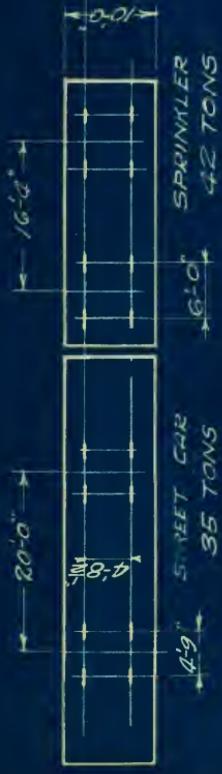


FIGURE "A"

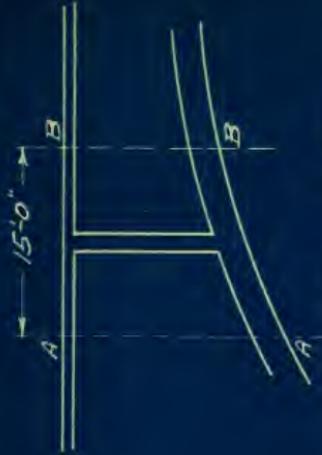


FIGURE "B"

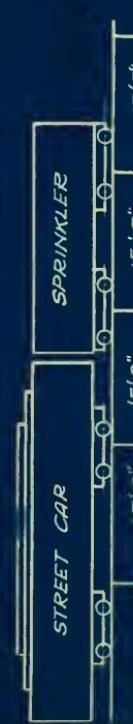


FIGURE "C"



FIGURE "D"

PLATE I



FIGURE A.
SIDEWALK BRACKET

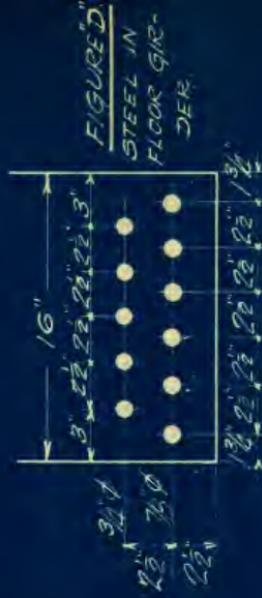


FIGURE C.

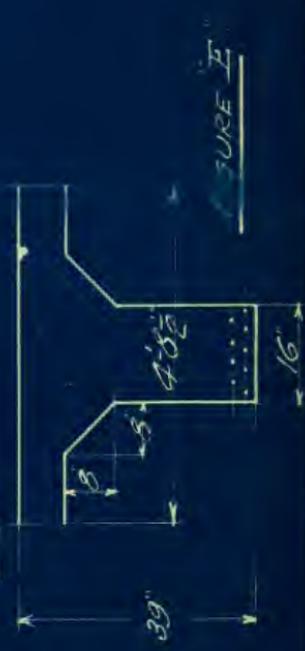
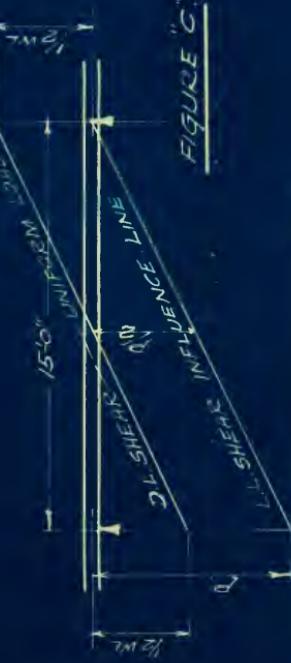


FIG. B



FIG. A

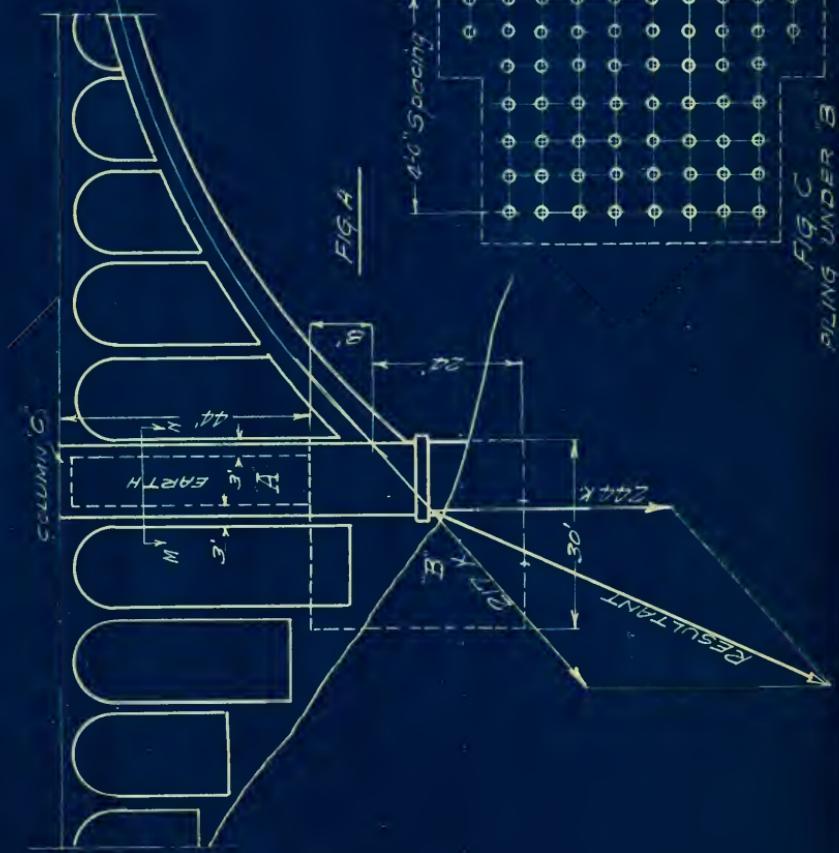


FIG. C
PLATE UNDER



TABLE A

PROPERTIES OF RETAINING EQUIL. DIVISIONS

NO. OF DIVISIONS

DEPTH	I_c	$15I_s$	$I = I_c + 15I_s$	$i = \frac{I}{I_c}$	i	i_{AS}	I	i
1	3.00	2.251	2.653	2.654	3.993	4.05	4.162	2.45
2	3.01	2.272	2.631	2.630	3.952	4.032	4.200	2.48
3	3.05	2.453	2.847	2.785	3.661	3.95	4.526	3.05
4	3.18	2.676	3.085	2.961	3.376	3.00	4.821	3.62
5	3.23	2.809	3.295	3.104	3.552	3.601	5.051	3.81
6	3.25	2.944	3.444	3.248	3.875	3.680	5.387	4.12
7	3.46	3.451	3.334	3.739	2.644	3.51	6.283	3.98
8	3.66	4.279	3.974	4.057	2.425	3.29	7.302	3.25
9	3.76	4.508	4.608	4.511	2.035	2.94	9.200	3.36
10	3.86	5.258	4.445	5.703	1.753	2.61	9.474	3.50
11	4.22	6.261	6.955	6.766	1.480	2.12	11.252	4.58
12	4.41	7.139	7.534	7.083	1.309	1.77	12.732	5.65
13	4.68	8.208	8.146	9.164	1.073	1.35	16.25	7.410
14	4.96	10.172	8.745	10.867	0.822	0.981	18.850	10.200
					3.4769		16.974	8.75

.5% STEEL ABOVE & BELOW MAX. LENGTH HRC = K.D = $\frac{45.672}{37.242} \times 14.72 = 11.657$.

$$\text{AVERAGE } i = \frac{\sum i}{N} = \frac{3676.9}{12} = 306.4$$

$$AS.i = 116.97 \times 306.4 = 36244 = 2.074.$$

(SEE PLATE 3).

TABLE "B"

Point	x	y	χ^2	γ^2	m_1	m_R	$[m_1 + m_R] \gamma$	$[m_1 - m_R] \gamma$
1	2.281	.0011	4.331	.0000	-	44.52	-	.069
2	8.500	0.202	39.199	.0408	-	96.01	-	38.784
3	13.200	.411	112.870	.109	-	165.00	-	131.520
4	14.50	.906	235.56	.521	-	223.3	-	440.86
5	20.584	1.521	415.52	2.3104	-	326.2	-	1204.446
6	25.840	2.340	681.60	5.4756	-	560.7	-	2624.076
7	31.590	3.502	991.96	12.250	-	701.22	-	5349.400
8	38.240	5.100	1466.49	26.0100	-	1082.2	-	1038.440
9	45.616	7.262	2126.68	52.707	-	1446.4	-	21088.796
10	52.150	10.260	3019.48	105.27	-	1999.3	-	41025.640
11	63.6	14.310	4266.18	202.49	-	2699.8	-	77274.0
12	72.92	19.72	5976.59	388.89	-	3600.5	-	141984.0
13	84.9	26.396	8335.48	696.74	-	4908.5	-	269/68.800
14	98.34	37.680	11653.0	1418.680	-	6535.6	-	493002.9
Σ	129.61	39324.83	2913.858		-	29070.46	-	7054.471.733

CASE I
SPIN COVERED ENTIRELY WITH LIVE LOAD

$H_o = 172.5$ KIPS
 $V_o = 0$
 $M_o = 132$ KIP-FEET

TABLE "C"

POINT	THRUST LEFT	ECCENTRIC DIST RIGHT	LEFT	BENDING MOMENT	RIGHT
1	174.8	174.8	+.558	+.558	+.87.65
2	174.8	174.8	+.407	+.407	+.71.21
3	174.8	174.8	.209	+.209	+.36.68
4	176.1	176.1	.207	+.267	+.74.0
5	176.1	176.1	.074	+.074	+.1.3
6	176.1	176.1	-.1165	-.1165	-.20.5
7	179.0	179.0	-.113	-.113	-.20.2
8	179.0	179.0	-.335	-.335	-.60.0
9	183.8	183.8	-.252	-.252	-.46.4
10	183.8	183.8	-.373	-.373	-.70.3
11	191.4	191.4	-.350	-.350	-.67.8
12	191.4	191.4	-.149	-.149	-.28.5
13	202.8	202.8	-.817	-.817	-.165.5
14	217.7	217.7	+.788	+.788	+.171.4
	ABOVE	GRAPHICALLY	SHOWN ON PLATE 5.		

CASE I
SPAN COVERED ENTIRELY WITH LINE LOAD

TABLE-C-2

POINT	e/h	$M/h^2 f_c$	f	f_c	$1/k$	f_s
1	.185	.107	597	98	.2210	
2	.134	.089	598	.84	.1851	
3	.1636	.057	532	.504	.1632	
4	.0872	.068	513	.610	.1584	
5	.0242	.0234	403	.240	.2460	
6	.0374	.0342	427	.320	.4840	
7	.0359	.0339	416	.318	.5020	
8	.1040	.0773	522	.700	.3718	
9	.0746	.0625	464	.560	.2050	
10	.10635	.0778	508	.710	.4925	
11	.0974	.0741	410	.682	.0120	
12	.0382	.0354	372	.320	.4238	
13	.1908	.1089	575	1.00	.2184	
14	.1644	.1012	519	.941	.3830	

$$f_s = \frac{C_{ASE}}{nf_c/I_1 - d\frac{I}{A_h}}$$

CASE: TURNER, A. MARSHALL
 PLATE: TOWER & CO.

TABLE "D"

P_T	χ	y	χ^2	m_L	m_{τ}	$[m_L + m_R]y$	$[m_L - m_R]y$
1	2.281	.001	4.331	.0000	-34.52	-34.52	.069
2	8.5	.201	39.199	.0205	-96.010	-96.010	.38784
3	13.2	.411	112.7	.169	-165.00	-165.00	.0
4	14.5	.906	235.56	.021	-243.3	-241.45	+440.123
5	20.584	1.551	415.62	2.310	-335.2	-375.4	-172.832
6	25.84	2.34	651.50	54.756	-500.7	-519.45	-2527.551
7	31.59	3.502	911.96	12.25	-766.2	-645.2	-4932.90
8	38.24	5.1	1466.49	26.01	-1082.2	-966.0	-10445.82
9	45.616	7.262	2126.68	52.707	-1397.3	-1207.25	-19199.233
10	52.15	10.26	3019.48	105.27	-1999.3	-1725.0	-38211.318
11	63.60	14.31	4266.18	204.49	-2694.8	-2294.7	-71485.625
12	72.02	14.72	5976.59	308.89	-3600.5	-3052.5	-131197.16
13	84.90	26.396	5335.45	696.74	-4908.5	-4139.6	-23786.7
14	98.34	37.68	11653.0	1418.68	-6538.6	-5502.0	-2553704.85
15	129.611	39324.83	2913.858	-24406.13	-19004.08	-43490.21	-971355.195
							+264229.029

CASE JLIVECNLEFTHILLONLY

$$H_c = 105.5 \text{ KIPS}$$

$$V_3 = 3.36 \text{ "}$$

$$M_c = 20.7 \text{ KIPS-FEET}$$

TABLE "E"

POINT	THRUST		ECCENTRIC DIST.		BENDING MOMENT		RIGHT
	LEFT	RIGHT	LEFT	RIGHT	LEFT	RIGHT	
1	165.6	165.4	-.0362	-.1288	-.4579	-.21.3	
2	165.6	165.4	-.279	-.4226	-.46.43	-.70.5	
3	165.6	165.4	-.1928	-.238	-.31.9	-.52.4	
4	166.7	166.6	-.1437	-.089	-.23.9	-.40.2	
5	166.7	166.6	-.328	-.434	-.54.7	-.72.3	
6	166.7	166.6	-.399	-.628	-.65.7	-.83.1	
7	169.5	169.0	-.338	-.476	-.57.3	-.76.6	
8	169.5	169.0	-.679	-.637	-.80.0	-.90.8	
9	174.8	172.9	-.1218	-.6265	-.21.3	-.108.1	
10	174.8	172.9	-.657	-.621	-.07.6	-.10.5	
11	182.6	181.1	-.521	-.663	-.95.1	-.118.4	
12	182.6	181.1	-.382	-.663	-.65.6	-.11.6	
13	184.0	186.8	-.915	-.176	-.151.3	-.32.5	
14	209.4	194.5	+.176	+.110	+.36.4	+.24.0	
ABOVE = $\frac{1}{2} \times 2104 \times 222.2$		SHOWS $\frac{1}{2} \times 2104 \times 222.2$		SHOWS $\frac{1}{2} \times 2104 \times 222.2$		SHOWS $\frac{1}{2} \times 2104 \times 222.2$	

CASE II
LINE LOAD ON LEFT HALF ONLY

POINT	e/h	$M_{bh}^2 f_c$	f_c	$\frac{f}{f_c}$	f_5
1	.0121	.0151	303	.120	4110
2	.0924	.0718	463	.642	3630
3	.0632	.0557	427	.461	3890
4	.0470	.0426	416	.391	4112
5	.062	.0995	401	.922	2810
6	.1294	.083	539	.818	2998
7	.1072	.0778	574	.710	3260
8	.1623	.0956	600	.930	2740
9	.0361	.0348	372	.311	4290
10	.158	.0981	561	.94	2480
11	.143	.0927	537	.862	3072
12	.098	.0741	432	.780	3262
13	.213	.1158	592	1.210	1820
14	.037	.0351	327	.310	3772

$$f_s = \frac{\text{CASE II}}{n_f F_I - d_{kh}'} [LEFT]$$

MAX TURBINE
RELATIVE TIME

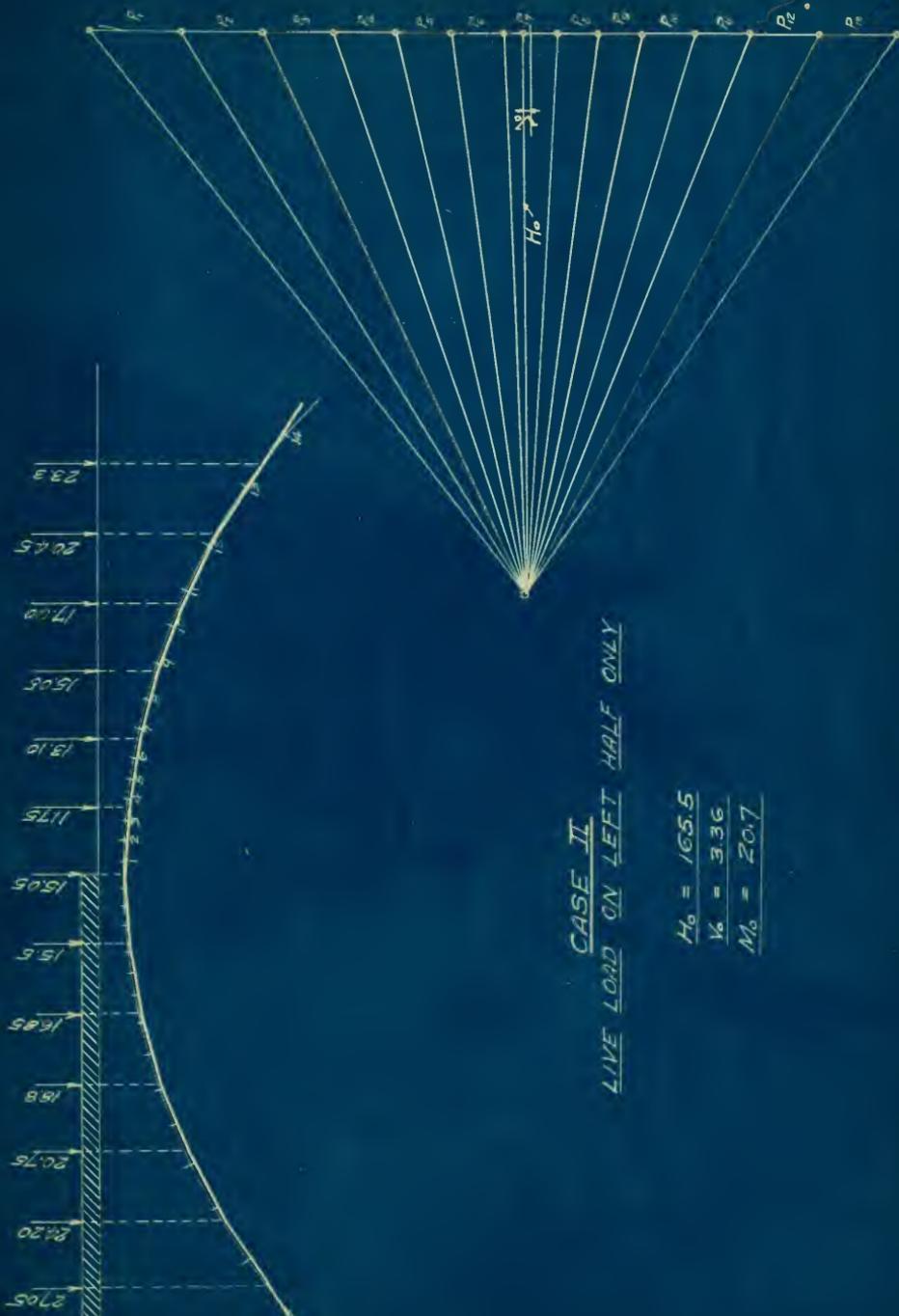
TABLE E-3

POINT	e/h	M/bh^2f_{c}	f	f_c	f_s
1	.0428	.0381	.428	.362	.4700
2	.1410	.0921	.582	.860	.3124
3	.144	.093	.579	.872	.3086
4	.158	.0992	.597	.895	.2960
5	.141	.093	.564	.860	.3240
6	.170	.103	.599	.950	.2594
7	.151	.0957	.588	.890	.2896
8	.168	.102	.567	.928	.3620
9	.185	.108	.602	.981	.2408
10	.178	.106	.571	.970	.2552
11	.182	.106	.609	.981	.2432
12	.017	.018	.295	.150	.3915
13	.0228	.0385	.335	.358	.2962
14	.232	.1197	.559	.1210	.1430

FROM TAN & MAYER
PLATES 200 & 100

$$f_s = \frac{CASE II}{n_f} \left[\frac{1}{L_f - d_f h} \right] [RIGHT]$$





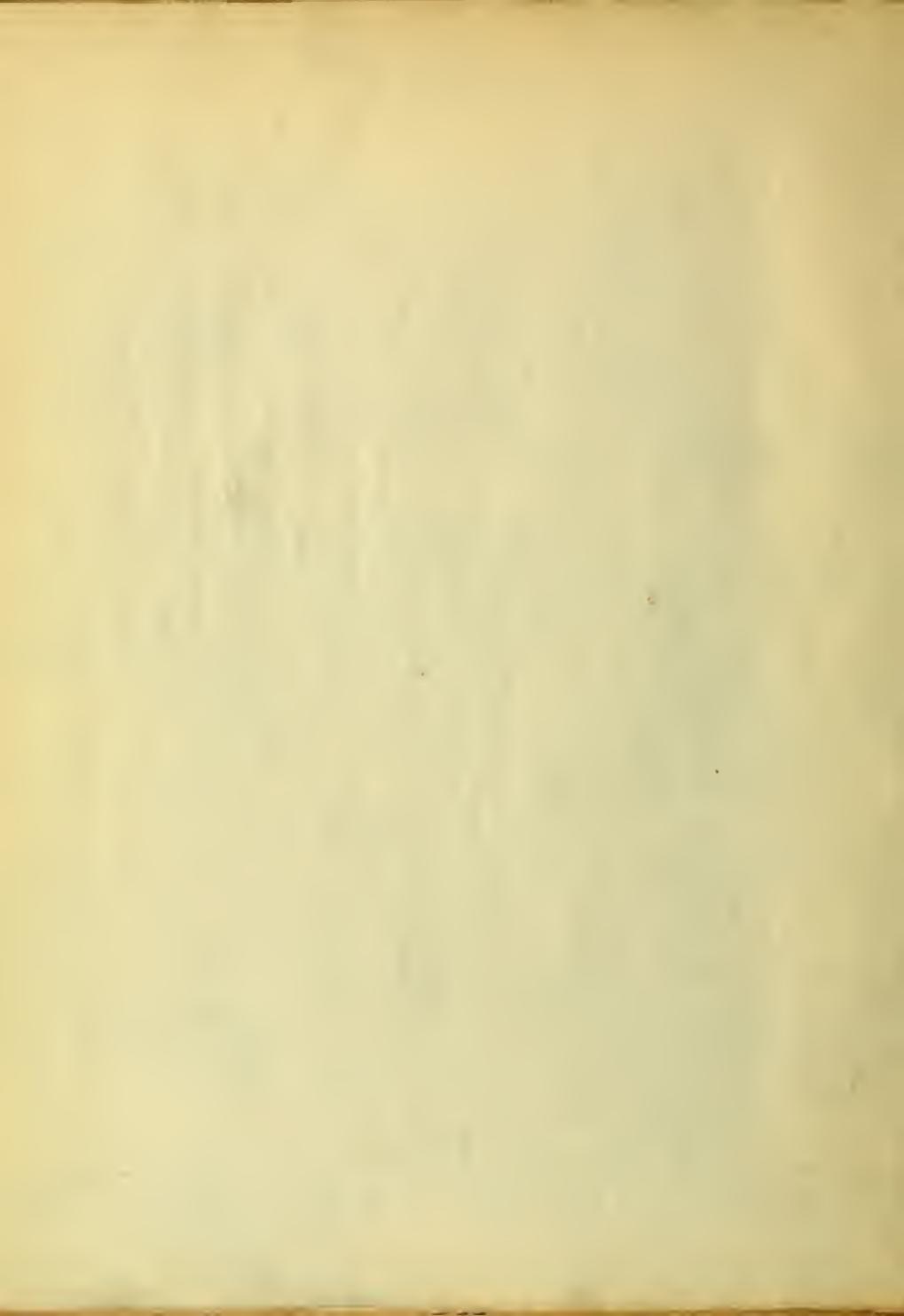
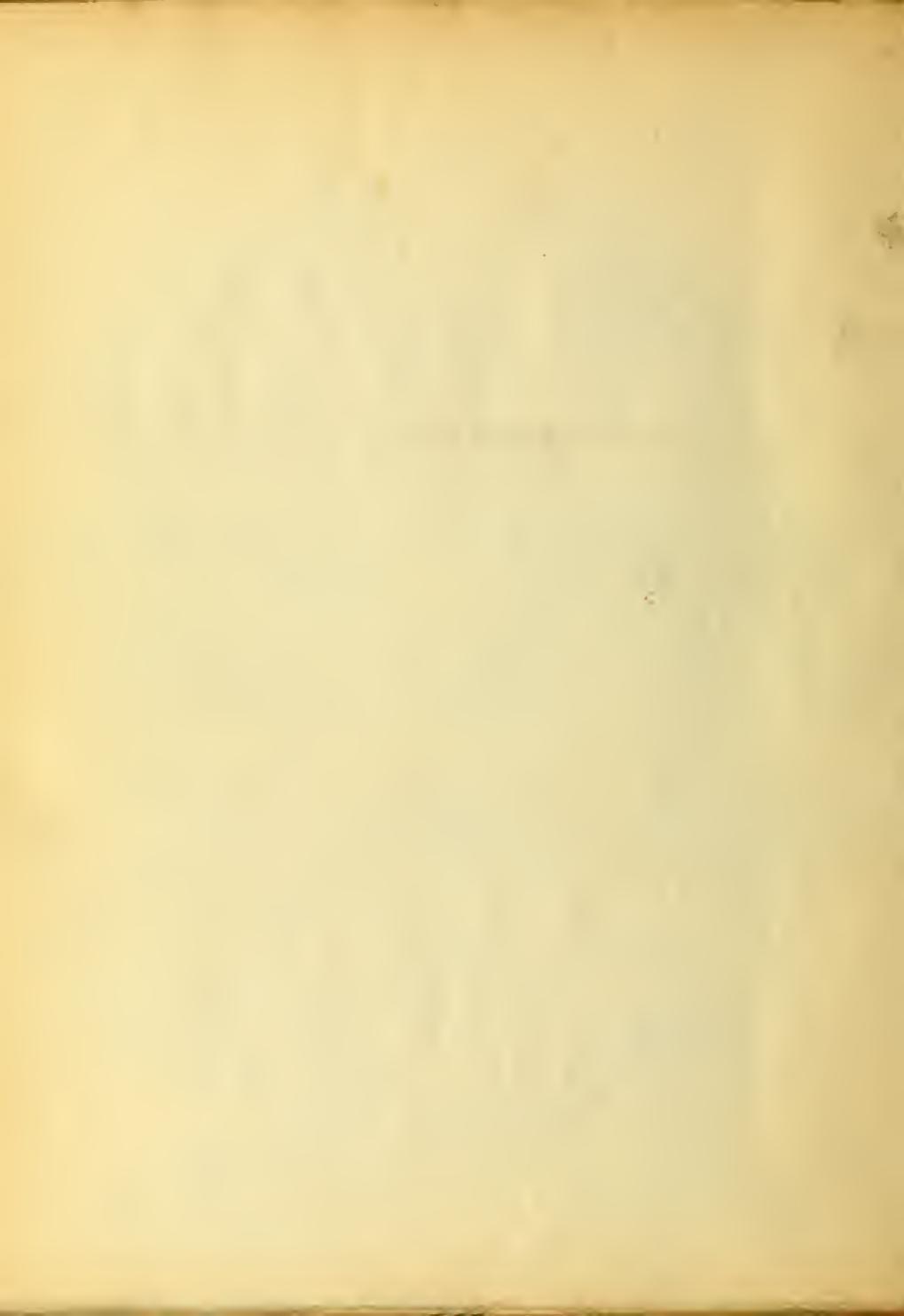


TABLE "F"

<u>PT</u>	<u>X</u>	<u>Y</u>	<u>X²</u>	<u>Y²</u>	<u>M₁</u>	<u>M₂</u>	<u>M₁₂</u>	<u>[M₁ + M₂] Y</u>	<u>[M₁ - M₂] X</u>
1	2.281	.001	4.331	.0000	-44.52	-44.52	-44.52	- .069	0.0
2	8.5	.201	39.199	.0408	-96.01	-96.01	-96.01	-38.782	0.0
3	13.2	.411	112.87	.169	-165.0	-165.0	-165.0	-131.52	0
4	14.5	.906	235.56	.821	-243.3	-243.3	-243.3	-140.86	0
5	20.584	1.521	415.52	2.310	-396.2	-396.2	-396.2	-1204.448	0
6	25.84	2.34	681.50	5.4766	-560.7	-560.7	-560.7	-2624.076	0
7	31.59	3.502	991.96	12.25	-764.2	-764.2	-764.2	-5349.4	0
8	35.24	5.10	1466.49	26.01	-1082.2	-1082.2	-1082.2	-11038.44	0
9	45.601	7.264	2126.68	52.707	-1443.6	-1443.6	-1443.6	-20961.072	0
10	52.15	10.26	3019.48	105.27	-1963.8	-1963.8	-1963.8	-40297.176	0
11	63.60	14.31	4266.18	204.490	-2615.1	-2615.1	-2615.1	-74844.162	0
12	72.92	19.72	5976.59	308.69	-3436.0	-3436.0	-3436.0	-13535.5.04	0
13	81.90	26.396	8335.48	696.74	-4610.7	-4610.7	-4610.7	-263444.96	0
14	98.34	37.68	11653.0	1418.68	-6066.9	-6066.9	-6066.9	-457201.90	0
Σ	129.611	39.324.83	2913.658		-46956.46			-973092.707	0

CASE III
LIVE LOAD ON MIDDLE THIRD
H₀ = 1567 KIPS
V₀ = 0
M₀ = 2264 KIP. FEET



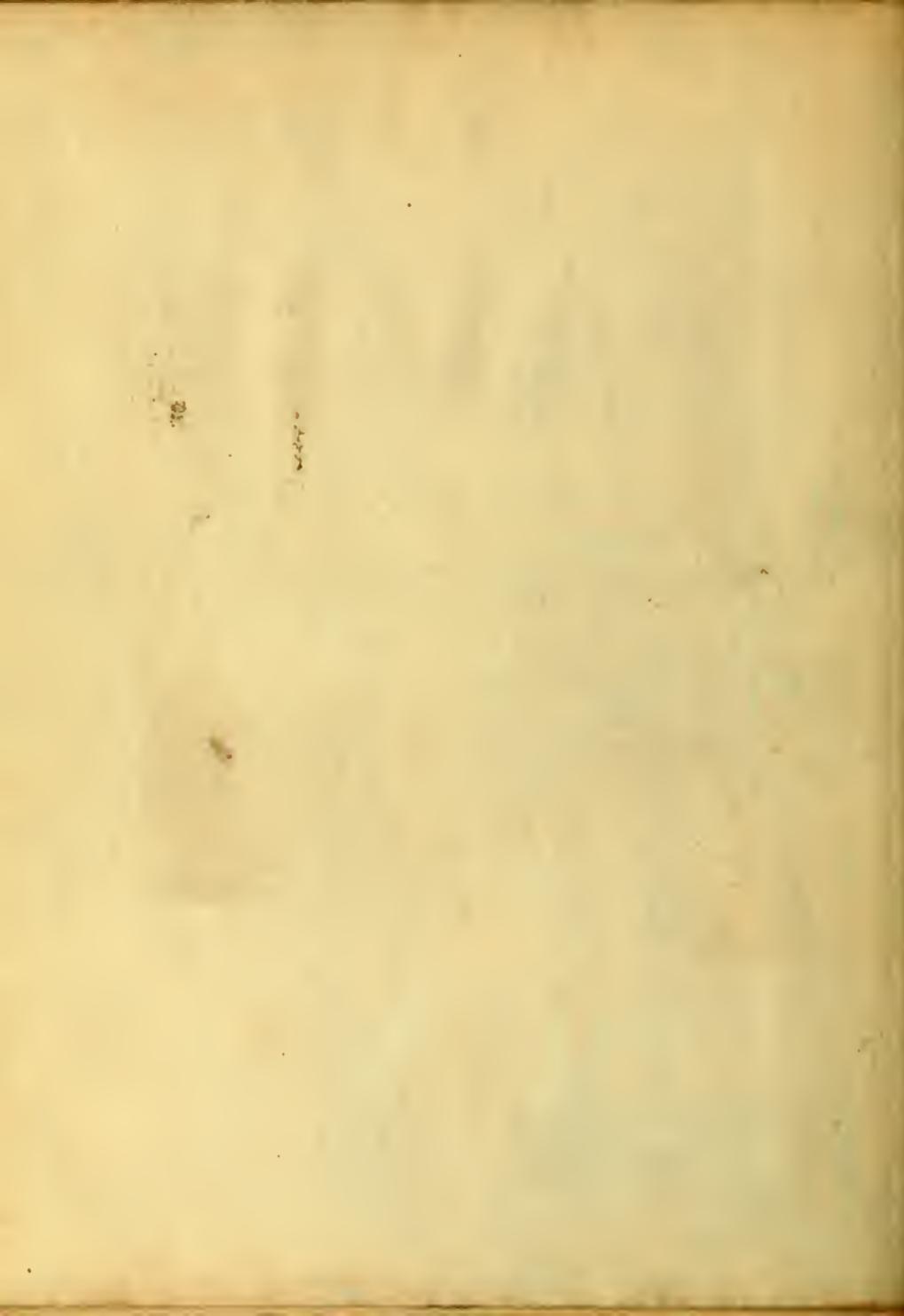
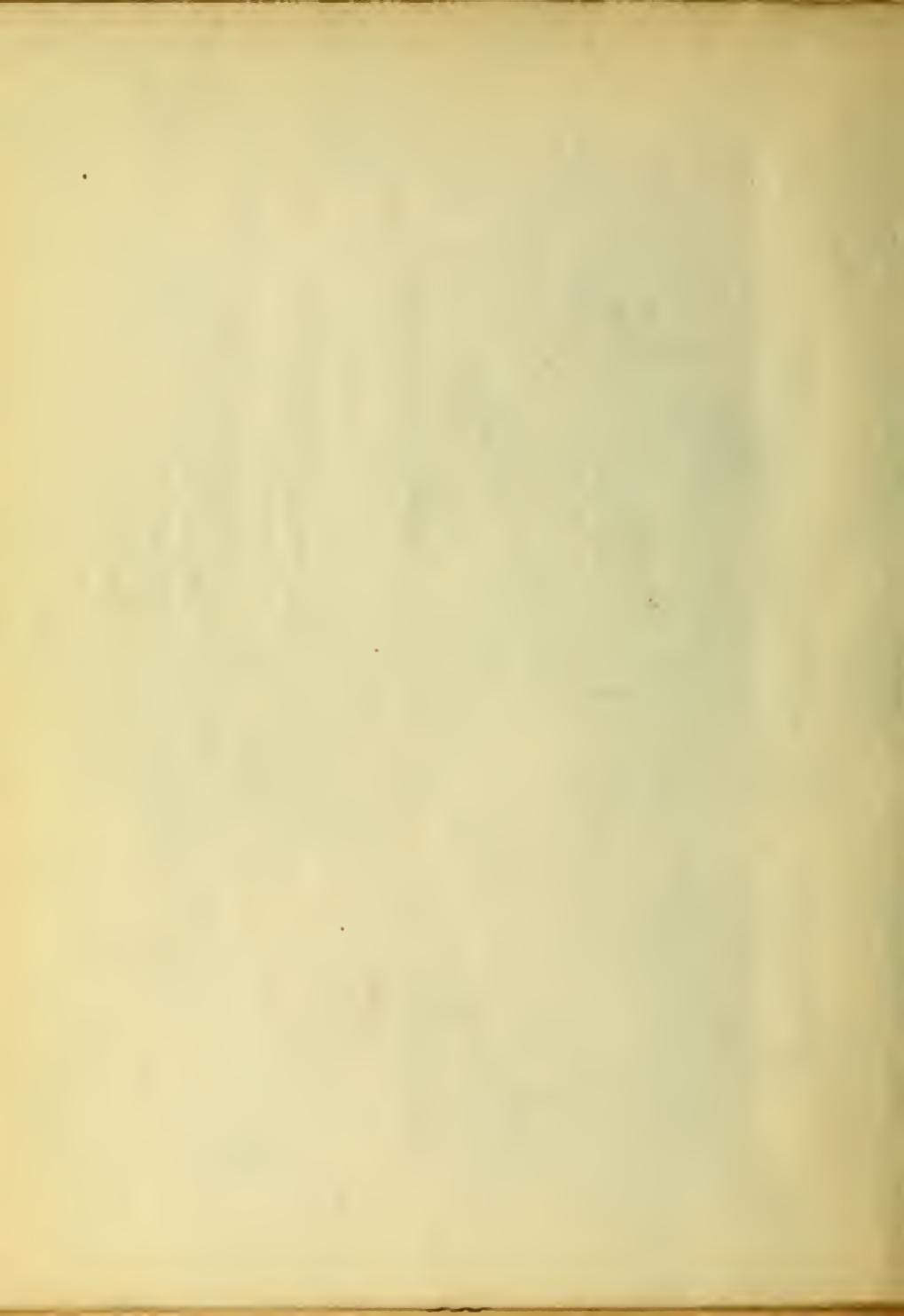


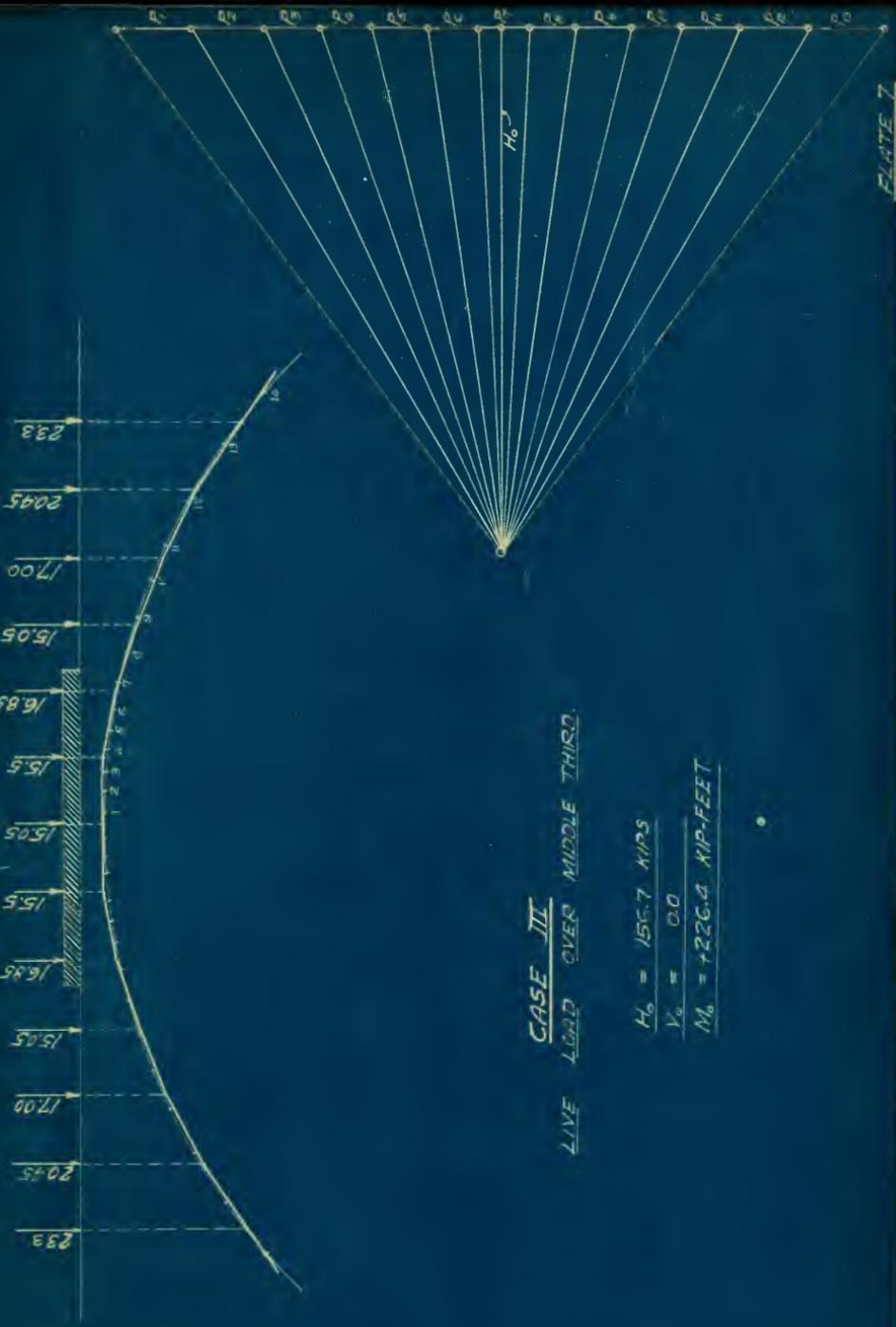
TABLE-G2

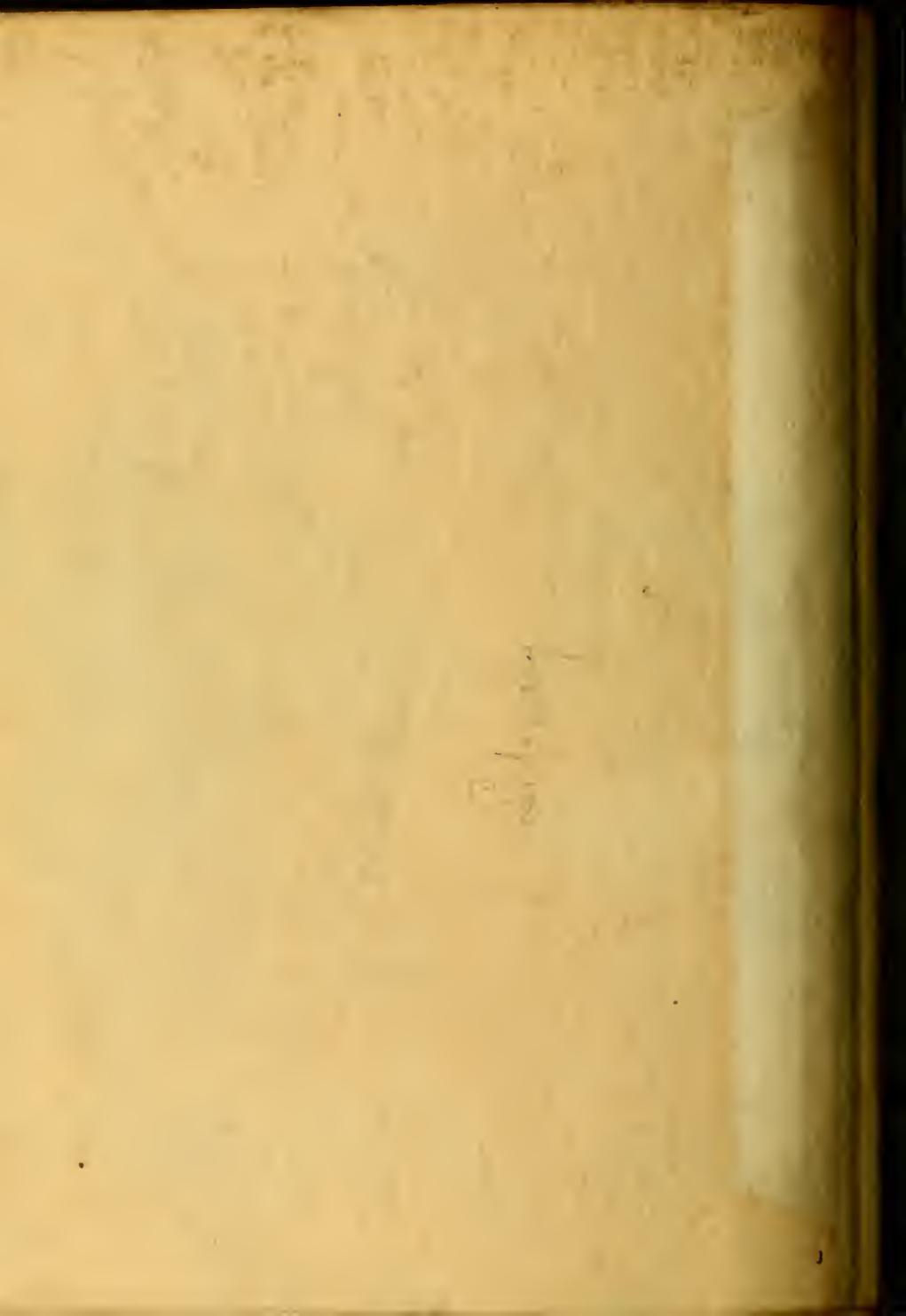
POINT	$\frac{e}{h}$	$\frac{M}{bh^2f_c}$	f_c	$1/k$	f_s
1	.941	.119	598	1.020	2230
2	.1634	.107	581	.930	2905
3	.140	.092	550	.860	2800
4	.203	.115	603	1.040	3004
5	.150	.095	557	.890	2785
6	.065	.056	418	.490	4120
7	.020	.022	339	.210	4400
8	.110	.078	492	.740	3250
9	.140	.092	525	.860	2953
10	.182	.105	586	.980	2510
11	.128	.112	608	1.101	2250
12	.161	.105	533	.981	1804
13	.227	.120	563	1.10	1680
14	.071	.059	350	.551	3150.

FROM TABLE G-1
P-175-300

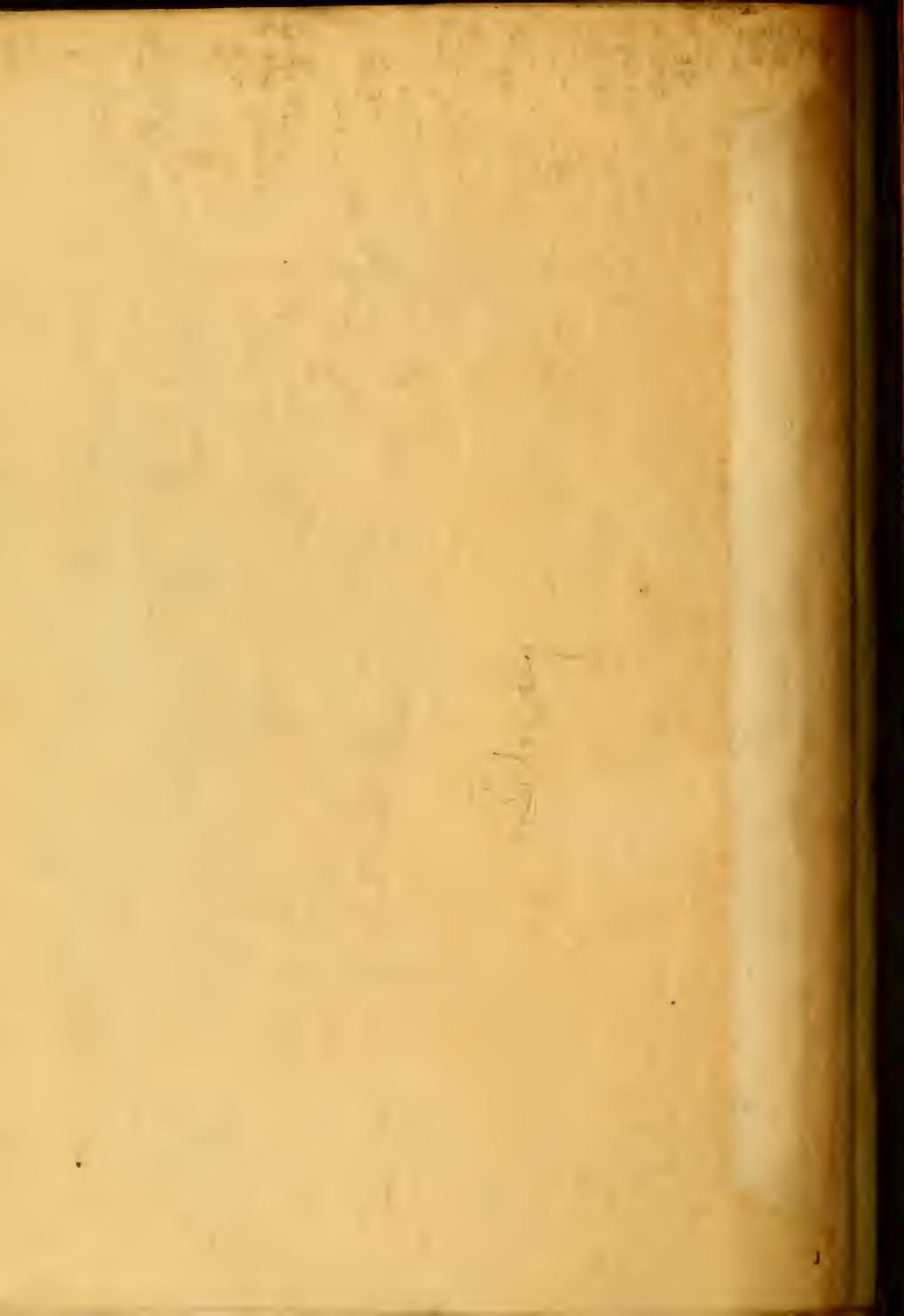
$$\begin{aligned} f_s &= m_f \left[1 - \frac{d}{h} \right] \\ f_c &\text{ from } \frac{M}{bh^2f_c} \end{aligned}$$



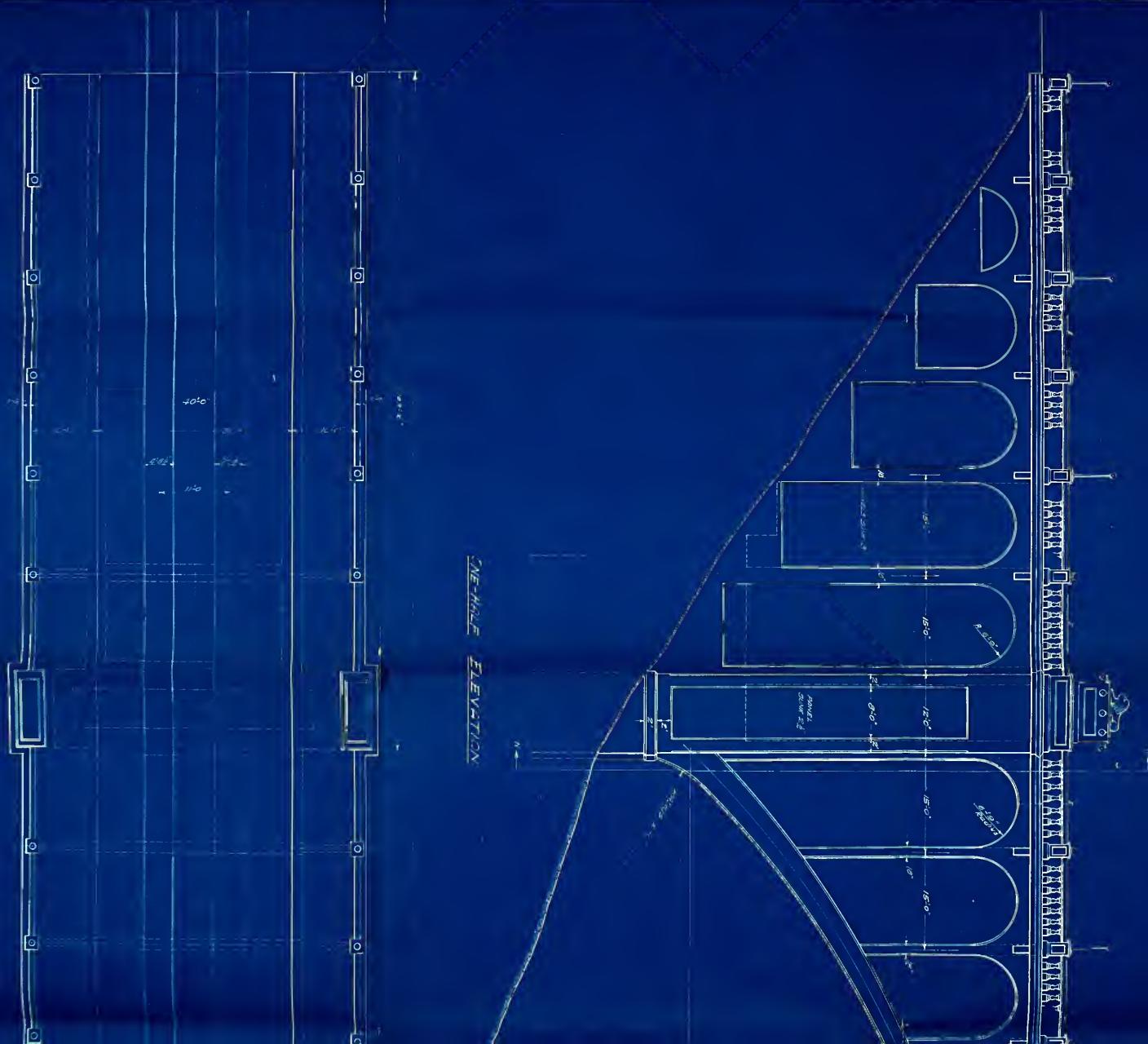








CHEMICAL PLANT

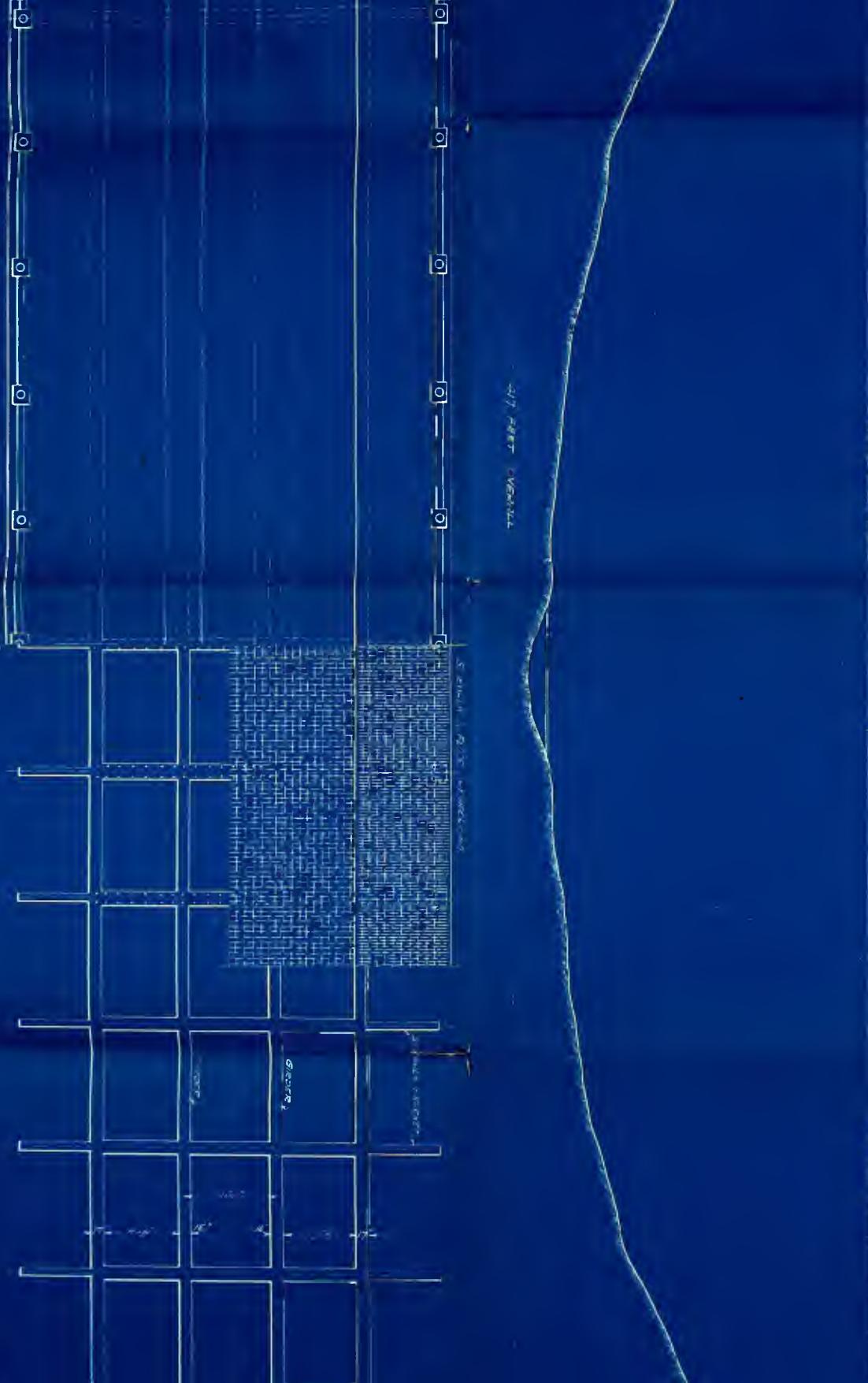
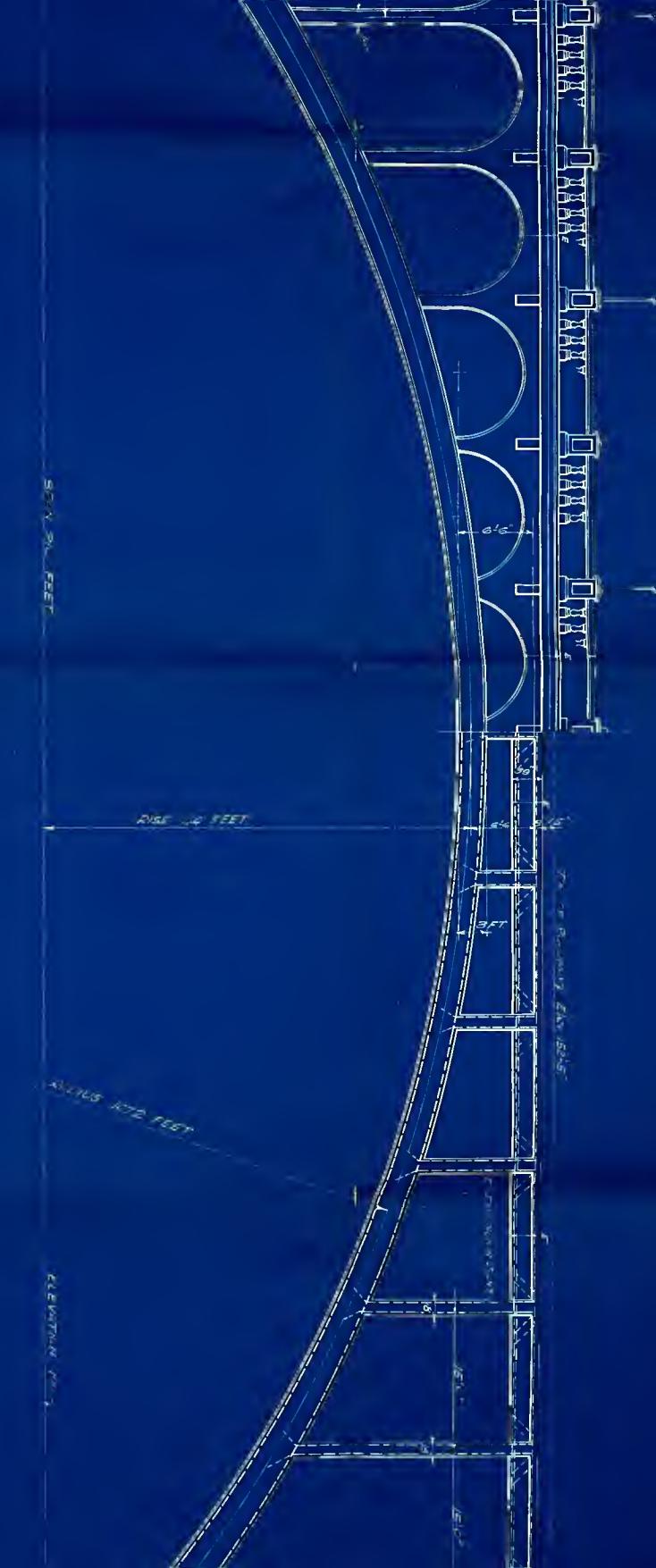


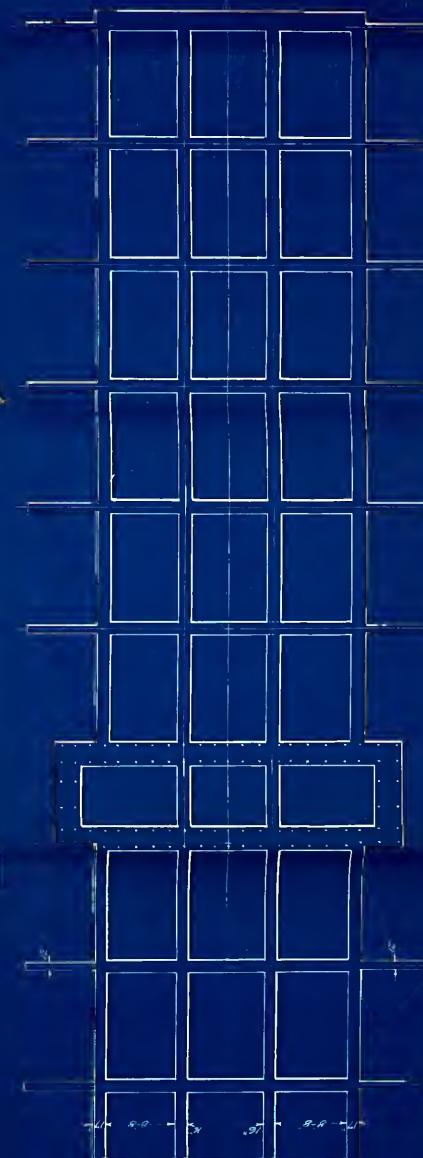
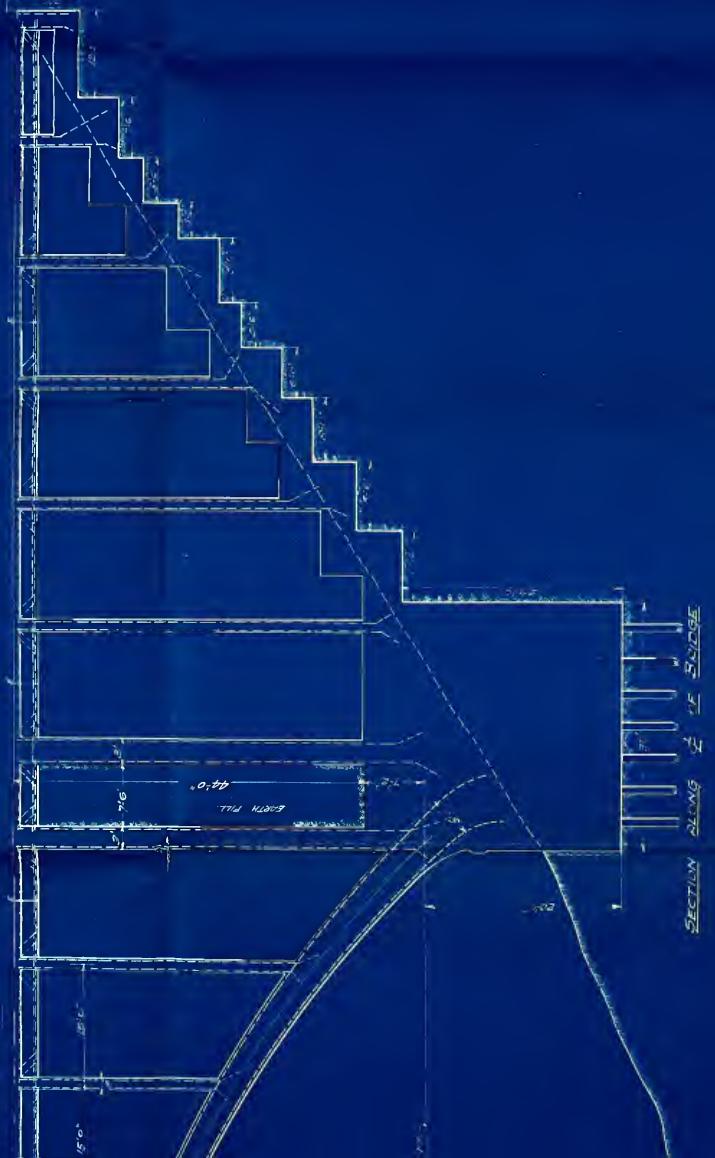
ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
TIE-SIS

OPEN SHANDEL REINFORCED CONCRETE ARCH BRIDGE
SPAN 210 FEET

SCALE $\frac{1}{3}$ INCH = 1 FEET

MAY 1911

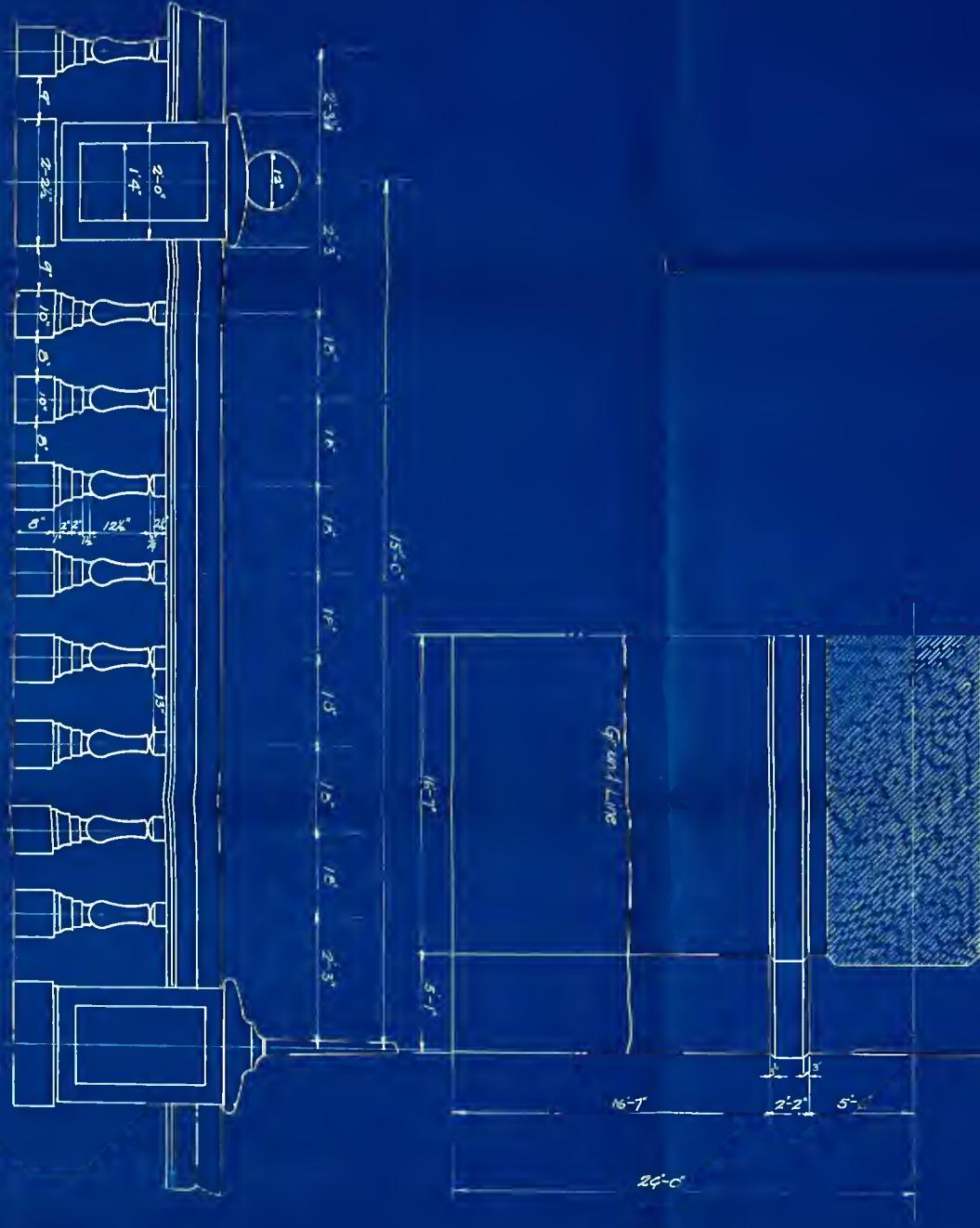




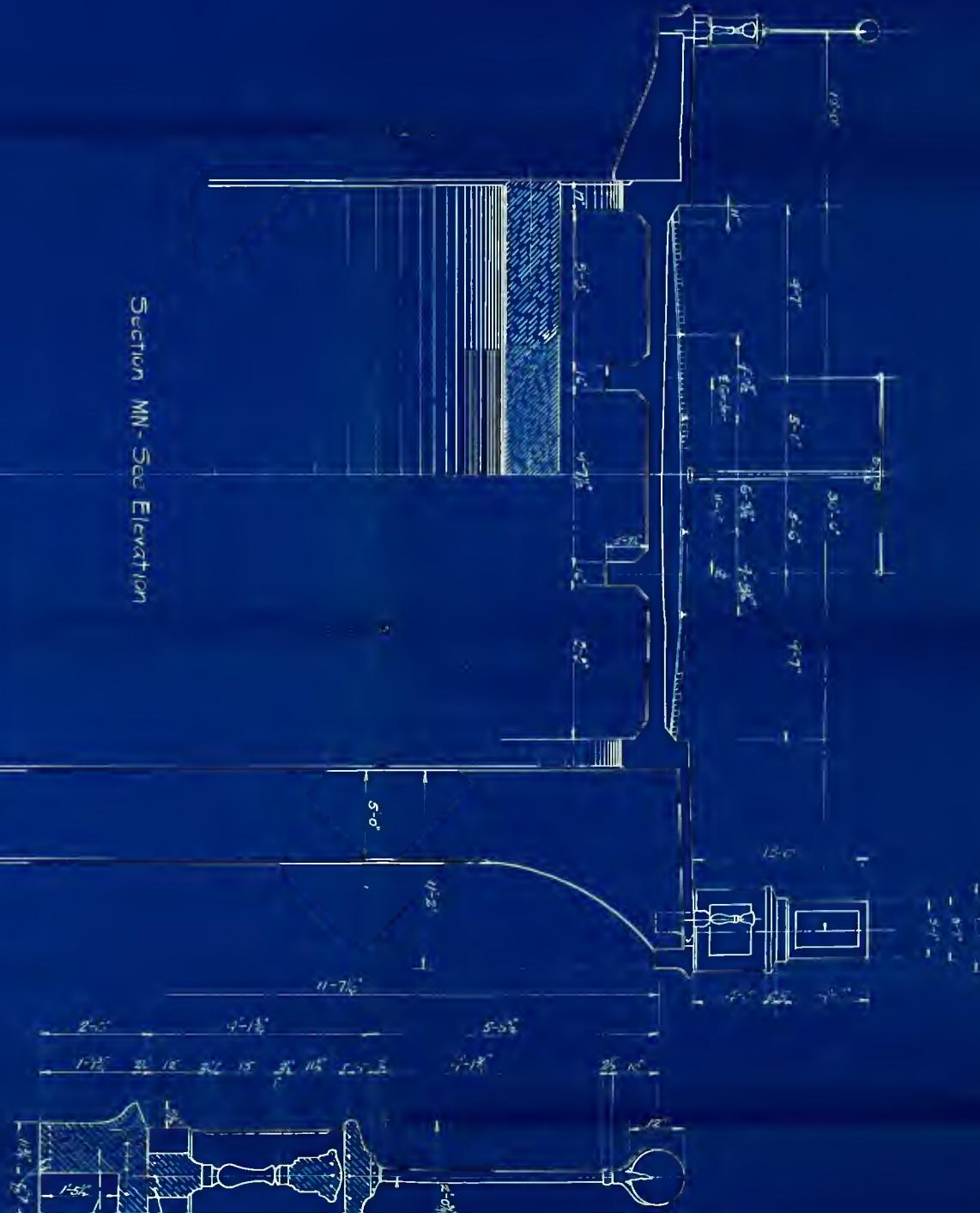
SECTION FLOOR PLAN

SECTION SEVENTH FLOOR

SECTION EIGHTH FLOOR



Section MN - See Elevation



THEESIS

OPEN SPANDREL REINFORCED CONCRETE ARCH BRIDGE
SPAN-216'0" RISE-4'-0"
MAY-1911 SCALES $\left\{ \begin{array}{l} \frac{1}{4}\text{-FOOT} \\ \frac{1}{4}\text{-FOOT} \end{array} \right.$
S. H. Mays
G. G. Barnes
E. M. Mandel

ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

